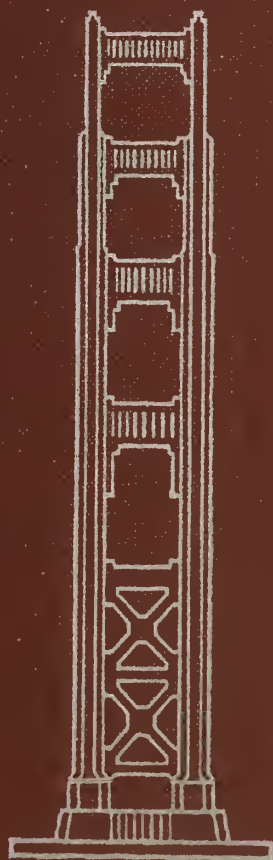


The GOLDEN GATE BRIDGE



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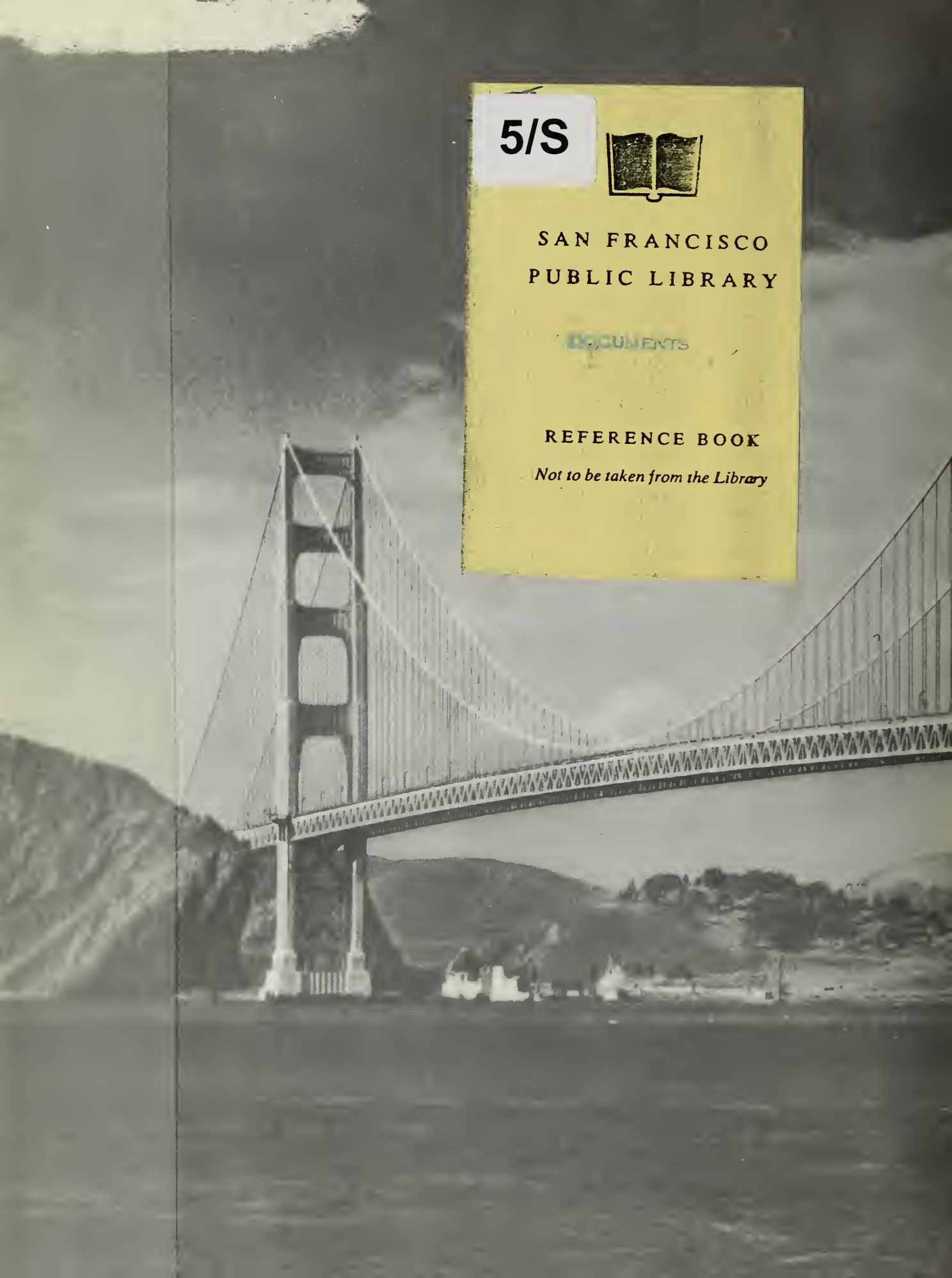


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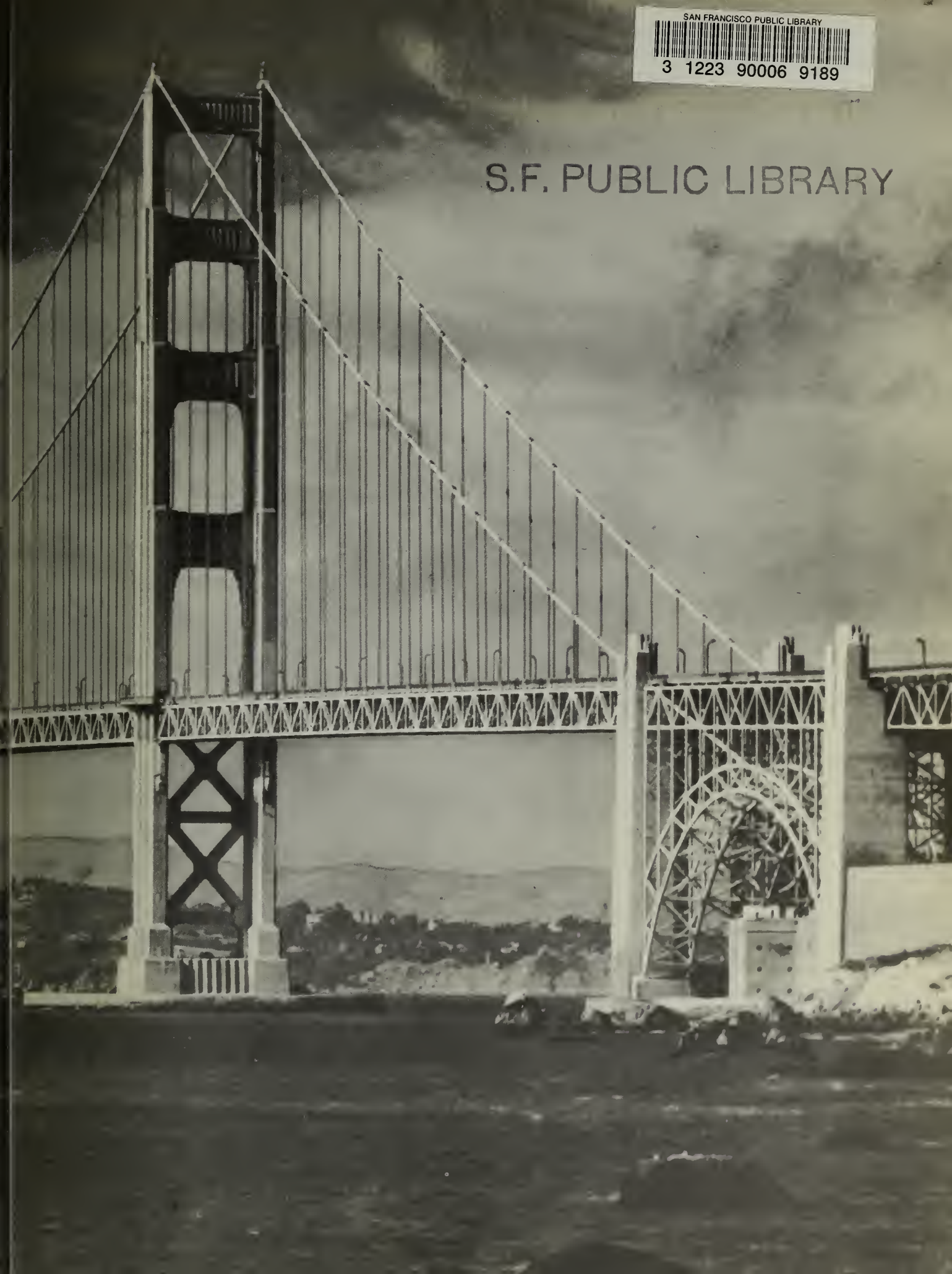
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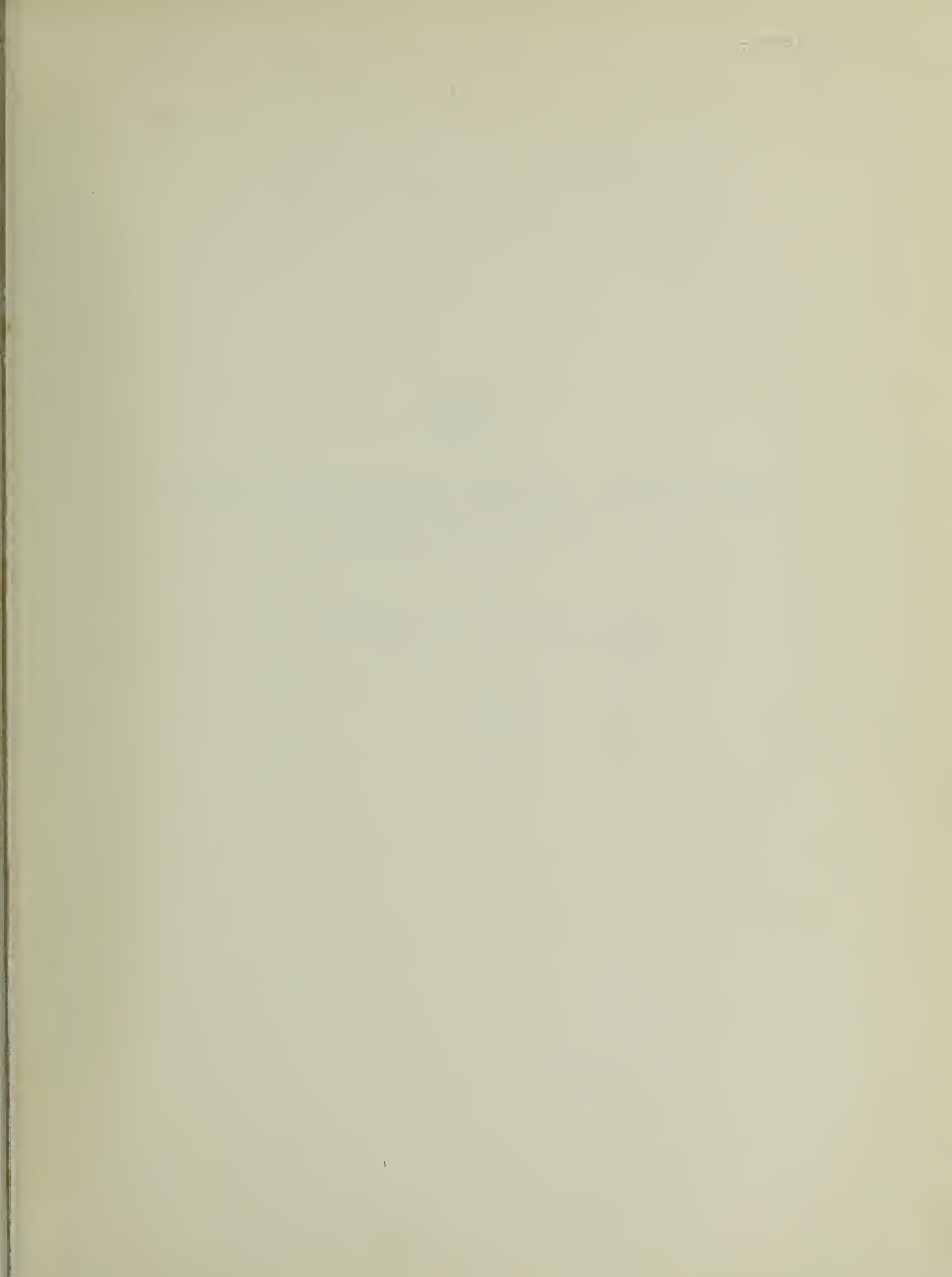
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THE GOLDEN GATE BRIDGE

REPORT OF THE CHIEF ENGINEER
TO THE BOARD OF DIRECTORS OF
THE GOLDEN GATE BRIDGE AND
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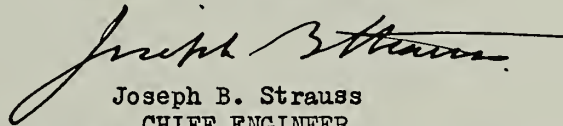
September 30, 1937

To the Honorable Board of Directors
Golden Gate Bridge and Highway District

Gentlemen:

I have the honor to transmit herewith the final report
of the Chief Engineer on the building of the Golden Gate Bridge,
covering the history of the project from its inception to its
completion and including a description of the technical and
other phases of the work.

Respectfully submitted,



Joseph B. Strauss
CHIEF ENGINEER
GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT

JBS-m



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BOOK ONE

GENERAL HISTORY AND RUNNING
STORY OF PLANNING, DESIGNING,
BUILDING THE WORLD'S LARGEST
BRIDGE SPAN. PRECONSTRUCTION
DIFFICULTIES AND OBSTRUCTIONS
ARE SURMOUNTED AND NOVEL
ENGINEERING PROBLEMS INCIDENT
TO THE BRIDGING OF THE GOLDEN
GATE ARE SOLVED.

GENERAL HISTORY

ABOUT seventy years ago, William H. Seward, Andrew Johnson's secretary of state and the man whose vision moved the United States to purchase Alaska, uttered this prophecy: "The Pacific Ocean with its shores—its islands and the vast region beyond—will become the chief theater of events of the world's great hereafter." Seward's prophecy is in course of fulfillment today, and, as the drama of the Pacific unfolds, the Great West Coast Region of North America is to play a leading part. That same region, with its system of transcontinental highways leading eastward, is in a larger sense the hinterland of the Golden Gate Bridge; and San Francisco by virtue of its strategic location is the natural commercial and banking center of that hinterland.

THE DISTRICT AND ITS CHARACTERISTICS

The Golden Gate Bridge is a project conceived and justified as a toll bridge. And since successful toll bridges have almost invariably been sustained by local patronage—for the local patron will use a bridge perhaps a dozen times each year while the itinerant tourist may cross but once—it will be appropriate first to examine the physiographic and political characteristics of the region more immediately contiguous to the project's bridge site. The remoter areas may be dismissed with the brief statement that the Pacific Slope is a region of high relief with limited coastal plain and generally north-south trending mountain ranges and valleys. Long-distance travel for pleasure therefore is confined by mountain barriers to the north-south routes. And since the interior valleys of California and Oregon are oppressively hot in summer, and the passes of the Siskiyou—a choppy cross range of indefinite trend at the California-Oregon line—are snowbound and hazardous in winter, such travel tends to seek the all-year-climate routes of the narrow coastal strip where the summers are cool, the winters mild and snow is virtually unknown. The Golden Gate Bridge links the north and south sections of these all-year coastal routes together.

PHYSIOGRAPHY: The San Francisco Bay Region is topographically unique. It consists of 463 square miles of saltwater-flooded basin, largest land-locked harbor in the world, the marginal occupiable lands about its shores, and the mountains round about, rising at some places to the maximum elevation of 3000 feet. The nine counties tributary to the bay, with their combined area of 4,750,000 acres, are embraced within the compass of this circumferential rim of mountains, and the present-day population of these counties is nearly 2,000,000 persons, owning 550,000 automobiles.

The rim is cleft at but two places: on the west where the gorge of the Golden Gate cuts through the coastal highlands and permits access to the tidal waters of the Pacific; on the east where chains of considerable altitude, which shut off the bay region from

the vast reaches of California's highly developed central basin, are cut by Carquinez Strait. The strait carries the discharge of California's two central basin rivers, the Sacramento and the San Joaquin.

Long rugged spurs thrust toward the bay from the circumferential mountains and divide the alluvial plain into extended valleys radiating from the marginal levels of the shore. It was along these valleys that the early settlements were made. Availability of fresh water, accessibility of the lands, and, generally, the possibility of water transportation to San Francisco determined their locations. The older towns will be found today on navigable water where they still serve as subports for the produce of the hinterland.

The entire occupiable area within the bay region, including plain and upland, is nearly 9,000,000 acres, or an area easily capable of supporting a future population of some 6,000,000 persons. Omitting lands not now strictly urban or suburban in character the district contracts to about 6,000,000 acres, and this contracted area is truly susceptible of development as San Francisco's tributary metropolitan district. It includes the entire city and county of San Francisco; the flat and rolling lands on the west side of the bay where the settlement now is almost continuous as far south as and including San Jose; the alluvial plain and adjoining uplands along the east side of the bay with its 25-mile fringe of compactly continuous East Bay cities; the terrain along the south shoreline of Carquinez Strait and Suisun Bay extending eastward to Antioch; the opposite northbank of the strait from Benicia westward, thence northerly to Napa and vicinity; the land west of Petaluma Creek and San Pablo Bay extending southward to the bridge site; and the towns north of Petaluma to and including Santa Rosa. Excluding Santa Rosa and its environs, this area is circumjacent to the tidal and navigable waters of San Francisco Bay and its saltwater-flooded extensions. It embraces more than ninety per cent of the population of the 9 bay counties, and it may be depended upon to contribute more than ninety per cent of the traffic over the Golden Gate Bridge until such time as the State of California will have built a proper highway link between the Sacramento Valley and the north approach of the bridge, and opened up the Inland Empire to the project.

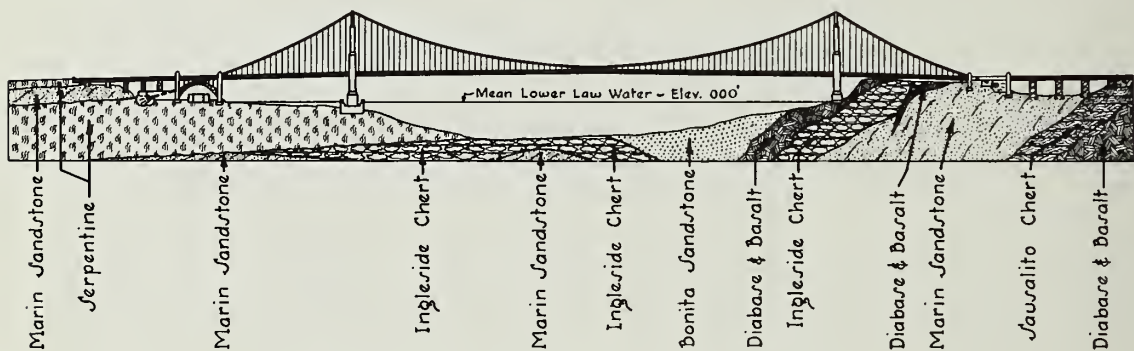
POLITICAL CHARACTERISTICS: It is natural that the development of an area such as we have described should exhibit considerable diffusion. Metropolitan growth elsewhere in the country has usually evolved more or less concentrically—as topography permitted—about the early settlement. For this reason metropolitan areas elsewhere are generally more compact and uniform in structure. Had it not been for the intervening bay and the intrusion of radial hilly spurs, both of which have acted as barriers to contiguous growth, there is no doubt that San Francisco's metropolitan district would have developed according to the conventional pattern. But although the individual settlements which constitute the bay region have of necessity spread out from the central hub more or less like the spokes of a wheel, the inhabitants of this cluster of towns and cities are as closely welded together by commerce, by social contacts and by habit of

thought as are the inhabitants of districts more compactly and uniformly developed; and the life of the people of the entire area is industrially, commercially and sociologically homogeneous to an exceptional degree.

GEOLOGY: San Francisco Bay was once a wide valley, part of the present-day system comprising the valleys of Santa Clara, Sonoma and Napa. The Golden Gate was then a river gorge through which the greater part of the drainage of California flowed as a fresh water stream to the ocean. Gradual subsidence, as it deepened the gorge, permitted the waters of the ocean to invade the valley till it became the 463 square-mile, saltwater-flooded area we have already noted.

The rocks traversed by the Golden Gate belong to a series of formations known to geologists as the Franciscan Series and comprise many different kinds of rocks, some sedimentary and some of igneous origin. These formations are widespread and extensively exposed throughout the coastal ranges of California.

After the Franciscan Series had accumulated to a thickness of a mile or more throughout a basin coextensive with the coastal ranges, the region was greatly disturbed



Geological cross-section of the Golden Gate on bridge alignment

by igneous intrusions. As a result, two general types of igneous rock are to be found in many localities. One of these is peridotite, now generally altered to serpentine, and the other, basalt. It happens that the south pier of the Golden Gate Bridge is founded on serpentine and the north pier on basalt. The fact that the narrowest part of the Golden Gate—which is the location selected for the bridge—is bounded on one side by basalt and on the other by serpentine, exemplifies the relatively great resistance of these rocks to the ordinary agencies of erosion.

Because of the intense internal shearing which takes place during its alteration by the process of hydration from peridotite, the tensile strength of serpentine is apt to be relatively low. But it is entirely adequate when confined—as it is. The load stress transmitted by the south pier to the foundation rock is less than 150 pounds on the square inch, and this is relatively a small load. This fact was practically demonstrated by the writer at Fort Point on a representative section of serpentine at sea level during his

preliminary investigation. The test section, 20 by 20 inches in area, was loaded to the extent of 92 tons, or 460 pounds per square inch, without yielding. Subsequent pressure tests were conducted in the inspection wells, which were built into the pier and used for access to the rock bottom for this purpose and for purposes of inspection. These tests, together with direct examination, fully confirmed the adequacy of the bedrock for the support of the pier.

It is reasonable to assume that once or twice in a century San Francisco will be shaken by a violent earthquake. The trace of San Andreas fault, upon which a sudden slip occurred in 1906 with disastrous results to San Francisco, lies six miles to the west of the bridge site. But there is no evidence that the Golden Gate itself is in danger of a dislocation such that differential movement between the two bridge-ends might be caused. Even when we contemplate the possible destruction of the Golden Gate Bridge by an earthquake of exceptional violence, it should be borne in mind that any earthquake so violent that it would destroy the bridge would also completely destroy San Francisco. Despite this, the city still continues to build and grow, and growth necessarily involves the erection of large and expensive structures.

EARLY HISTORY OF THE SITE

Historians tell us that the early navigators, led by such sixteenth century worthies as Cabrillo and Drake, had sailed past the Golden Gate for 200 years without suspecting the existence of the great land-locked harbor; and that it remained for a party traveling overland to make the discovery. One Caspar de Portola, it seems, had been dispatched from Mexico to civilize the Indians and establish Spanish authority in California. He was en route to Monterey Bay to found a Mission. Missing his objective, he sent his sergeant, Ortega, on ahead to explore the coast, and, on November 1, 1769, Ortega unexpectedly came upon the entrance of the bay of San Francisco. Six years later, Lieutenant Ayala sailed his good ship "San Carlos" through the "uncharted narrows" and opened the Golden Gate to the commerce of the world. Ayala's was the first ship of record to anchor in the harbor.

Then came Captain Juan Bautista Anza, redoubtable soldier of Old Spain, under orders to establish a permanent settlement on the bay. He sailed through the Golden Gate in March, 1776, and, after planting a cross on the narrow bench where Fort Point now stands, selected sites for the Presidio and Mission Dolores. Padre Fout, Anza's secretary, declared the harbor to be a marvel of nature, "the port of ports".

But the establishment of great cities does not come about by directed processes, but fortuitously and by the shaping of events. It happened that the city did not grow up at Anza's Presidio, as he had hoped, but some four miles eastward at Yerba Buena Cove under the lee of Telegraph Hill, where the waters of the bay came up to what is now Montgomery Street. To this sheltered cove, boats from the Missions around the bay brought hides, tallow and produce to trade with the sea-going ships that found safe



anchorage there; for the opposite mainland shores were exposed to the gales, shoaled and fringed with tide flats.

The little village of Yerba Buena, which took its name from the cove, prospered exceedingly. It remained Yerba Buena until January 10, 1847, when Washington Bartlet, an American and alcalde at that time, changed its name by executive order to San Francisco. Thus the great city of the bay region, its site having been decreed by the exigencies of commerce, grew up at the tip of a narrow 40-mile-long peninsula, invested on three sides by water and otherwise cut off from its natural mainland trade area by mountain barriers on the south, instead of on the opposite-lying mainland shores.

Even the coming of the first transcontinental railroad in 1869, with its terminus four miles east across the bay at Oakland's waterfront, failed to shake San Francisco's preeminence. Other railroads entered the Bay Region. Ferry services multiplied with the coming of the railroads. Commuting by rail and ferry began to be a settled habit by 1900; but the automobile was still many years away.

Following the arrival of the first transcontinental railroad, some began to dream about bridging the main bay and its tributaries. The Southern Pacific's Dumbarton rail crossing over the narrows of the lower bay, a low-level, single-track structure, was built in 1910. Freight could now be routed to San Francisco via the round-about route over Dumbarton crossing. Oakland, however, still remained the terminus for mainline trains. Other proposals followed, many of them impractical and all of them relating to the bridging of the main bay, Carquinez Strait or the lower San Joaquin. But no definite plan had been submitted for bridging the Golden Gate until the publication of this writer's original plan of 1919. This plan opened the door and out of it grew the activities which gradually made the city bridge-minded. We shall have occasion to describe that plan more in detail later.

The rapid industrial growth of the San Francisco Bay Region dates from the opening of the Panama Canal in 1915. The Canal, almost overnight, brought about profound readjustments in the world's trade routes; the Pacific Coast and the Atlantic Seaboard became close neighbors; and California, once the most distant land on earth from the centers of white civilization, suddenly found itself at the cross-roads of the world's commerce. About five years later, the automobile began to be a factor to be reckoned with as a means of transportation.

DEVELOPMENT OF NEED FOR THE BRIDGE

San Francisco has been a complacent city, "supreme, indifferent to fate". Its rolling hills and intervening valleys have fostered a certain sectionalism and discouraged community thinking. We have seen that its growth and importance as a world city have come about through the operation of extraneous forces, and, we are to suppose, with little conscious effort on the part of the inhabitants.

Future historians will record that this self-sufficiency was rudely shaken by the rise of the automobile with its insistent demand for better highways, and that San Francisco

did not begin to assume its proper place as the natural center of the great Bay Region Metropolitan Area until archaic ferry services had given place to modern bridges. It will be appropriate, then, to trace the effect of the rise of the automobile on bay-crossing facilities.

In the year 1919, following the World War the volume of automobile travel across San Francisco Bay was relatively small compared with the tremendous present-day movement. The two ferry lines then operating out of San Francisco—one to Sausalito in Marin County across the Golden Gate, and the other to the foot of Broadway, Oakland, were controlled by an unprogressive rail monopoly. The service, grudgingly vouchsafed and infrequent, was rendered by rebuilt boats of ancient vintage, and the tolls charged were excessively high. A growing and insistent patronage was clamoring even then for relief, but somnolent operating officials gave no evidence of an awareness that real service was needed and must some day, somehow be provided.

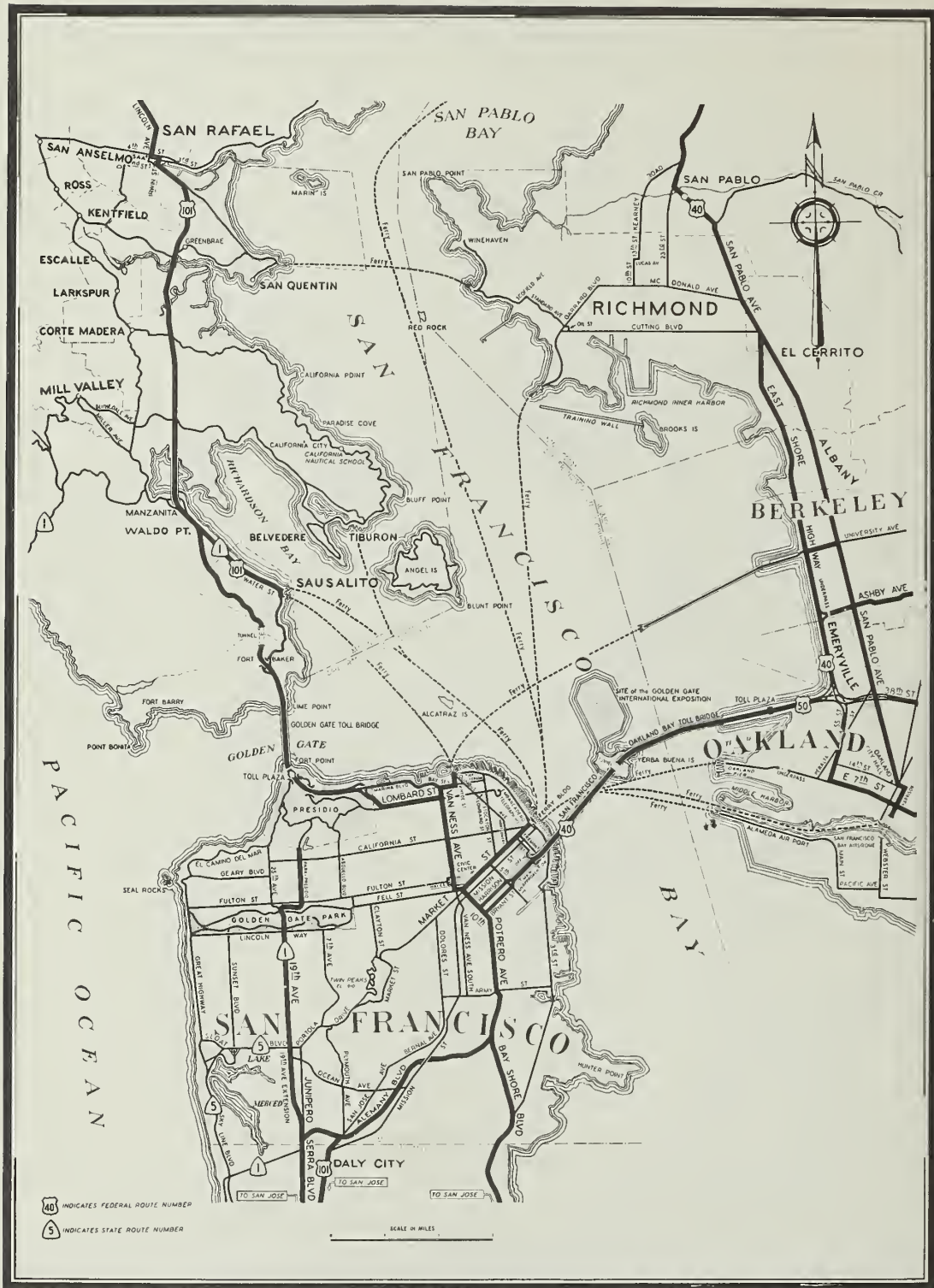
It remained for enterprising outsiders, who invaded the monopoly's field in 1921, to perceive the need and prove that fast, efficient Diesel-powered boats, designed exclusively for the transportation of automobiles, were demanded by the mounting traffic. The invaders promptly reduced tolls and the new lines prospered exceedingly. Belatedly, the monopoly, having acquired one of the competing lines by purchase meanwhile, improved its service, reduced tolls and recaptured a part of the profitable business. By 1929, motor vehicle ferry travel to and from San Francisco had increased more than seven-fold, and in May of that year, all of the services were taken over by interests associated with the monopoly and merged. The toll level, however, remained unchanged, for the interests in control were opposed to concessions as well as to progress. We shall have occasion later to mention those interests again, in connection with obstructive litigation and activities initiated by them by which they sought to delay the building of the Golden Gate Bridge.

It has been a characteristic of the San Francisco Bay vehicular travel that every expansion of ferry service in the past has been met at once by increased response on the part of the motoring public, and the volume of traffic induced by the expanded services has soon built up to the limit of the capacity of the new equipment. Service naturally improved somewhat following the merger, for the pooling of the fleets made possible the immediate diversion of extra boats during periods of peak flow to points where traffic pressure for the moment was greatest. But the capacity of the lines taken over at the time of the merger had already begun to be overtaxed long prior to May, 1929. Even the pooled fleets were wholly inadequate to cope with the normal week-end and holiday rush.

Travel across the Golden Gate is exceptionally heavy during week-ends and holidays. Maximum hourly offerings to the Marin ferry services of upwards of 3000 vehicles during unusually heavy holiday peaks had already been observed long before the opening of the Golden Gate Bridge, and the utmost capacity of the services, even under stress of dire necessity with all possible spare boats diverted thereto, was little more than a



Map of San Francisco Bay region



Main route and bridge connections in and out of San Francisco

thousand cars per hour. The highways leading to the Sausalito ferry plaza, therefore, were often jammed for miles by lines of irate motorists returning of a Sunday night from an outing in the northern counties.

Now the utmost capacity of San Francisco's combined fleet of vehicular ferries is but two thousand cars per hour, one way. This divides, one thousand north to Marin; one thousand east to the East Bay cities. A thousand cars per hour is approximately the capacity of an old-time, horse-and-buggy road, but even such a road is faster than the ferries and the automobilist using it would not have to wait for boats. It was this type of traffic facility that had isolated San Francisco as respects the northern peninsula and eastern mainland and that San Francisco had endured long after high speed roads and high-speed traffic facilities had become general.

FIRST STUDY, RECONNAISSANCE AND ESTIMATE

For some time the possibility of a bridge across the Golden Gate had been considered. In 1918 a resolution was introduced by the then Supervisor Richard J. Welch, now Congressman and a Director of the Bridge District, authorizing and directing a survey to be made by the City of San Francisco in order to determine the physical conditions at the site. During the same year the matter was brought to the attention of the writer by the city engineer, M. M. O'Shaughnessy, who made inquiry as to whether the writer would be interested in the solution of the problem. He accompanied this inquiry with the statement that "Everybody says it can't be done and that it would cost over \$100,000,000 if it could be done".

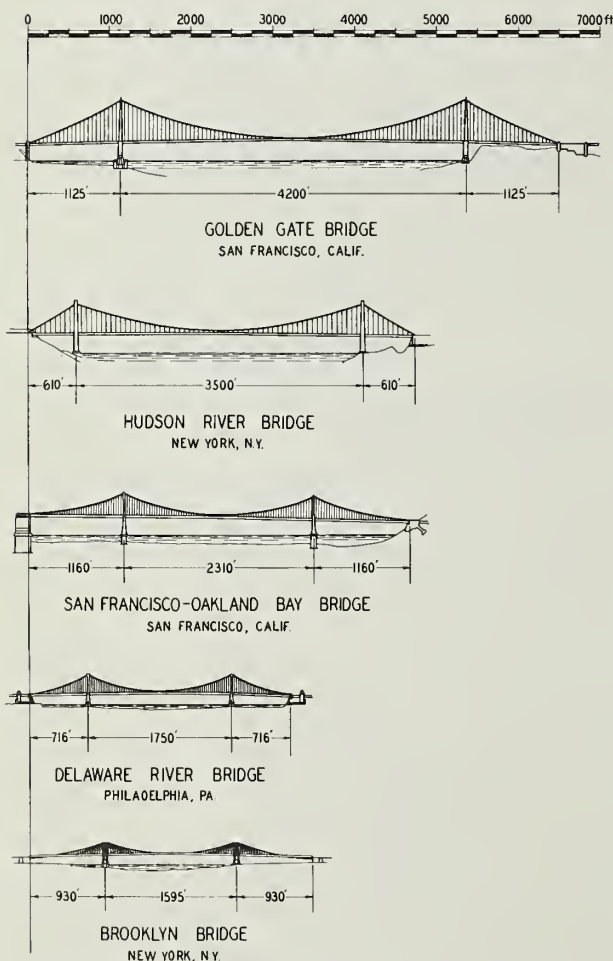
The writer thought it could be done and said so, and thus was initiated the first serious attempt to break through San Francisco's encircling water barriers. At that time, as we have noted, except for the Southern Pacific's single-track, low level railroad bridge across the Dumbarton narrows, neither San Francisco Bay nor any of its arms had been bridged, and all bay traffic was handled by a private monopoly which failed to meet the rapidly expanding needs of the community. This first practical proposal to bridge the Golden Gate altered all this. It stimulated local bridge-mindedness as it unfolded and became the inspiration for all subsequent bridge-the-bay projects.

At the Golden Gate the problems presented were many and difficult. Old precedents would have to be set aside, new ones established. We have seen that the channel is more than three hundred feet deep, and 5357 feet wide at its narrowest part, where, for cogent physical reasons having to do with foundation conditions and length of bridge, the structure would have to be located, leaving little room to spare for adjustments of alignment. The conditions indicated a main span length of about 4000 feet, and at that time spans of such great length had not been contemplated. In addition, the locality is subject to fog and high prevailing winds. It is exposed to the sweep of ocean storms and heavy swells from the Pacific, and the tidal current reaches the velocity of seven knots.

The site lay entirely within two military reservations—the Presidio on the south and

Fort Baker in Marin County on the north—so that the bridge approaches would interfere seriously with important military works, buildings and roads. The entrance to a great harbor had never been bridged, and there was a deep-seated objection to the establishment of such a precedent. There were strong military and naval objections on the ground that an enemy might by bombing the structure bottle up the harbor. The minimum permissible vertical clearance would be greater than that of any other bridge over navigable water. These considerations and the unprecedented size of the various component units of the structure, the vast quantities of materials to be assembled and routed to their rather inaccessible place of use, the new and original methods of construction to be devised for building a structure of such great magnitude virtually in the open sea, and, finally, the question of doing all this within practical limits of cost—all made the challenge to the engineer a formidable one.

Then, too, there were questions by no means less difficult of solution having to do



Spans from the world's notable suspension bridges drawn to the same scale for purpose of comparison with the Golden Gate span

with the financing of a project of this nature, and the legislative steps to be taken in order to create a political entity for building an inter-county structure. And finally, any movement set afoot to bridge the Golden Gate would without doubt be violently opposed by well-organized, powerful vested interests who had long enjoyed without question a monopoly of Bay Region transportation. We shall presently see that such opposition did develop, with a vehemence and persistence unique in the annals of bridge construction.

Following a brief personal inspection and reconnaissance of the site and its approaches, the writer returned to the city engineer naming two conditions under which he would undertake the task: first, a determination by the City of the limit of cost which would have to be met, and, second, the assembly by the City of the necessary sounding and topographical data fundamental to the making of such a study. The answer to the first condition was \$25,000,000. The answer to

the second was the survey already authorized by the Board of Supervisors under the resolution by Supervisor Welch. As already stated, Supervisor Welch had previously given such a project consideration and the Welch resolution was the first official action in the movement for a bridge to span the Golden Gate, and the resulting survey became the basis of the first Strauss design.

The survey was duly carried out with the cooperation of the United States Coast and Geodetic Survey. In May, 1920, the results thereof were transmitted by the City of San Francisco with request for proposals not only to the writer alone, as he had expected, but also to two other engineers. One appears not to have responded; the other, replying two years later, gave as a minimum cost \$60,000,000 and a maximum of \$77,000,000.

The writer's preliminary sketch and estimate were transmitted June 28, 1921, and embodied a symmetrical cantilever suspension design with 1320-foot anchor arms and a 4010-foot channel span, the latter being a composite of two 685-foot cantilever arms with a 2640-foot span of the conventional suspension type between them. The vertical navigational clearance was 200 feet at the center, the towers 800 feet high, and the 80-foot-wide single deck provided space for two surface tracks, four traffic lanes and two 7-foot sidewalks. The estimated cost was \$21,000,000 for foundations and superstructure, \$4,500,000 for highways and structural approaches, \$1,500,000 for engineering and administration, or a total of \$27,000,000. Interest during construction was not included in these figures.

EVOLUTION OF THE GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT

The writer's proposal, which was submitted jointly to City Engineer O'Shaughnessy and the Mayor's able and progressive secretary, Edward Rainey, was received with enthusiasm; for here at last was a feasible conception—a structure that could be financed and built for a sum within the limits set down as reasonable. At a first meeting between Mr. Rainey, Mr. O'Shaughnessy and the writer, the difficulty arising out of the fact that



The cable as it enters the saddle at the tower-top

Marin Municipal Water District. Mr. Coombs took charge of the legislative procedure at Sacramento and, aided by the officers and directors of the Association, the service and business organizations, the civic bodies and the press, the Act became a law on May 25, 1923. The Bridge District as finally constituted comprises the counties of Del Norte, Sonoma, Marin and San Francisco and parts of Napa and Mendocino. (See District map and brief summary of the Act.)

The services that Mr. Hotchkiss rendered the Association were invaluable. He gave it not only his own time but the necessary financial support, and, together with the writer, carried the principal burden during the long and trying preorganization period. The full story of that period is an epic which has no place in this report, but it would be unjust not to permanently record the great contribution made by that small company of early pioneers—Congressman Welch, Mr. Hotchkiss, Mr. Rainey, Mr. Doyle, Mr. Shannon, Captain Hibbard, Assemblyman Coombs and all those who labored so long and faithfully and purely as a public service in those days when the Golden Gate Bridge was still just a dream and when so many sought to keep it so.

On the following pages will be found a brief summary of the Golden Gate Bridge and the Highway District Act, which created the Golden Gate Bridge and Highway District and by which it functions.

BRIEF SUMMARY OF THE BRIDGE AND HIGHWAY DISTRICT ACT OF CALIFORNIA

The Golden Gate Bridge and Highway District was incorporated under the provisions of an act of the Legislature of the State of California approved May 25, 1923 (Stats. 1923, page 452; Deering's General Laws of California, 1931, Act No. 936). The original act of 1923 was amended by the state legislature in 1925, 1927 and 1931. Some of the essential features of the Bridge and Highway District are as follows:

FORMATION: The proceedings for the formation of the district include the adoption of ordinances by counties proposing to unite for the purpose of forming a bridge and highway district and the circulation and filing of petitions for the formation of the district within such counties. The petitions are filed with the Secretary of State, who is required to give notice of a time and place at which protests against the formation of the district may be filed. Protests, if any, must be submitted to the Superior Courts in the respective counties from which they were filed and after a hearing judgment is rendered by such Superior Courts, from which appeals may be taken to the State Supreme Court. If the final judgment is in favor of the validity of the district, the Secretary of State issues a certificate of incorporation, declaring the district with the boundaries as finally established to be duly incorporated as a bridge and highway district.

DIRECTORS: The district is governed by a board of directors, the members of which are appointed by the various boards of supervisors of the different counties comprising the district. Each county is entitled to representation and the number of directors to be appointed from each county varies in proportion to the population. The board of directors selects one of its members to act as president and appoints a secretary and other officers of the district, including a general manager, auditor and attorney.

POWERS: A bridge and highway district has perpetual existence, may sue and be sued, may acquire, construct, maintain and operate bridges, roads and other properties of a revenue producing character. The district has the right of eminent domain, may borrow money and issue bonds and cause taxes to be levied and collected.

BONDED INDEBTEDNESS: In order to incur a bonded indebtedness, the directors must submit a proposition to the electors of the district at a special election called for that purpose. Such election can be held only after due notice, and it requires the vote of two-thirds of the qualified electors to authorize the issuance of the bonds. No bonded indebtedness can be incurred in excess of fifteen per centum of the assessed value of all property in the district. The maximum term of the bonds must not exceed forty years and the maximum rate of interest must not exceed six per centum per annum. All such bonds are declared to be legal investments for savings banks and may be used to secure the deposit of public funds. The bonds may be made registerable as to principal alone, or as to both principal and interest.

TAXATION: If the revenues of the district are inadequate to pay the principal or interest of the bonded indebtedness as it becomes due, the board of directors is required to cause a tax to be levied upon all of the taxable property in the district, in addition to all other taxes levied for county or city and county purposes. In the month of July of each year, the board of directors is required to determine the amount necessary to be raised by taxation. It must also fix the rate of taxes to be levied which will raise the amount of money required by the district. The rate is then certified to the board of supervisors of each county within the district, with the direction that at the time and in the manner required by law for the levying of taxes for county purposes, such board of supervisors shall levy and collect a tax at the rate so specified and determined, and it is made the duty of the county or city and county officers to levy a tax so required and to collect the same in the time, form and manner that county or city and county taxes are collected. Such taxes shall be a lien upon all property within the territory comprising the district and of the same force and effect as other liens for taxes, and their collection may be enforced by the same means as provided for the enforcement of liens for county taxes.

SUPPORTING LEGISLATION AND JUDICIAL CONFIRMATION OF THE ACTS LISTED: The State Legislature by an act approved April 10, 1929, validated the Golden Gate Bridge and Highway District (Stats. 1929, page 165), and by a comprehensive validation act of March 12, 1931 (Stats. 1931, page 77; Deering's General Laws of California, 1931, Act No. 938) validated the formation and organization of all bridge and highway districts, the boundaries thereof, the proceedings of the boards of directors, all bonds and the proceedings for the issuance of such bonds, and the levy and collection of taxes for their payment. The constitutionality of the Bridge and Highway District Act, the proceedings for the formation of Golden Gate Bridge and Highway District and the issuance of its bonds, and the power of the District to levy and collect taxes upon all of the taxable property in the District, was upheld by the Supreme Court of California in the cases of *Doyle v. Jordan*, 200 Cal. 170; *Wheatley v. Superior Court*, 207 Cal. 722; *Dempster v. Superior Court*, 207 Cal. 795; *Crawford v. Superior Court*, 207 Cal. 797; *Esaisa v. Superior Court*, 207 Cal. 796, and *Golden Gate Bridge and Highway District v. Felt*, 82 Cal. Dec. 683. An appeal from the decision in the Crawford case to the Supreme Court of the United States was dismissed by that court for want of a substantial federal question (1930, 281 U.S. 692, 50 Sup. Ct. 238, 74 L. Ed. 1121). The United States District Court for the Southern Division of the Northern District of California, in *Del Norte Company v. Filmer et al.* (In Equity No. 3174-L), and *Garland Co. v. Filmer et al.* (In Equity No. 3109-K) again overruled the objections raised by certain taxpayers, under the Constitution of the United States, to the formation of the district, the proceedings for the issuance of the bonds and the right of the district to levy and collect taxes.

Immediately following the passage of the Act, steps were taken by the writer to submit an application to the Secretary of War for a permit to build the bridge and make use of the Federal property necessary for the proper location and maintenance of structures and approach roads. In view of the current rumor that no such permit would be granted, and since without a permit the project would have failed, it was essential that the application should be made prior to the formation of the District. The

application was made in the names of the counties of Marin and San Francisco jointly, and heard before Colonel, now General, Herbert Deakyne in San Francisco in May, 1924—the City of San Francisco, City Engineer O'Shaughnessy and the Bridging the Golden Gate Association assisting and the writer presenting the case. The testimony in favor of the project was overwhelming and, after due deliberation, Secretary of War Weeks issued a provisional permit December 20, 1924, granting authority to proceed pending the submission of later, definite plans. This grant, as became evident later, definitely committed the Government and made the project possible.

The reader will have noted that the Act provides among other things for the financing of the project by the issuance of bonds secured as to interest and principal primarily by the tolls derived from the bridge, supplemented as a secondary resource by the power of the District to levy taxes to make up deficiencies of operating revenue. Before the component parts of the District could be put together, therefore, and incorporated as a responsible political entity, it was necessary first to give the taxpayers who objected to the inclusion of their properties full opportunity to protest. And this was a made-to-order opportunity for the project's enemies. Taking full advantage of the situation by means of taxpayers' suits, the opposition, well-organized and with plenty of money to spend, launched a vigorous campaign of obstruction. For nearly six years the District was dragged through the courts of the various counties in succession. During these years of litigation Humboldt County first joined in the project and then withdrew and parts of Mendocino did the same. In fact all of Mendocino County would have withdrawn but for the courageous fight made by Mr. A. R. O'Brien, publisher of the Ukiah Republican Press, who later became a Director of the District, and who has always taken a leading part in the furtherance of the best interests of the bridge.

Mr. George H. Harlan conducted the litigation in behalf of the District, with the writer as key witness and aided by experts he brought from Chicago and New York. The opposition had as its witnesses reputable engineers who stated under oath that the project would cost a total of \$112,344,788 and that the piers and anchorages alone would cost \$28,800,500. These estimates when compared with the Chief Engineer's estimate and the actual cost as hereinafter mentioned will be of special interest. The opposition was represented by vigorous counsel, who left no stone unturned to prevent the formation of the District. Nevertheless the District prevailed in all the suits, and finally on December 4, 1928, was incorporated.



ORGANIZATION OF THE DISTRICT

The Board of Directors of the newly incorporated Golden Gate Bridge and Highway District was seated on January 23, 1929. The Board comprised the following:

	<i>County</i>
William P. Filmer, director and President.....	San Francisco
Henry Westbrook, Jr., director****	Del Norte
Robert H. Trumbull, director and Vice-President.....	Marin
A. R. O'Brien, director	Mendocino
Thomas Maxwell, director	Napa
Carl A. Henry, director*	San Francisco
Warren Shannon, director	San Francisco
William P. Stanton, director**	San Francisco
Francis V. Keesling, director**	San Francisco
Richard J. Welch, director	San Francisco
Frank P. Doyle, director	Sonoma
Joseph A. McMinn, director	Sonoma

The first act of the Board was the appointment of the following officers:

Alan MacDonald, General Manager, *** 1929 to 1933	San Francisco
Joseph B. Strauss, Chief Engineer, October 1929 to October 1937	San Francisco
George H. Harlan, Attorney, 1929.....	Marin
John R. Ruckstell, Auditor, * 1929 to 1935	San Francisco
William W. Felt, Jr., Secretary, 1929.....	San Francisco

Later the Board was increased to fourteen by the addition of the following two members:

George T. Cameron, *** 1930 to 1933	San Francisco
Harry Lutgens, 1930	Marin

Vacancies created by the deaths and retirements noted above were filled as they occurred by the following appointments:

John P. McLaughlin, director, 1934.....	San Francisco
Hugo D. Newhouse, director, 1933	San Francisco
Arthur M. Brown, Jr., director, 1934	San Francisco
William D. Hader, director, 1936	San Francisco
James Reed, General Manager, 1933 to 1937	San Francisco
Roy S. West, Auditor, 1936	San Francisco

*Died. **Retired at expiration of term. ***Subsequently resigned.

****Mr. Westbrook at the expiration of his first term was succeeded by Milton M. McVay, who in turn was succeeded by Mr. Westbrook.

The Board of Directors met on Wednesdays three times monthly throughout the construction period and functioned through appropriate committees. The two committees having the most direct contact with construction were the Building Committee, under the efficient chairmanship of Mr. Keesling up to the time of his retirement, when he was succeeded by Mr. Maxwell, an able leader, and the Highways, Roads and Traffic Committee, with its fearless and dynamic chairman, Mr. O'Brien. The Building Committee served as the liaison medium between the Board and the Chief Engineer and under the splendid leadership of Mr. Keesling was a most constructive force. The Finance Committee has been ably headed by Mr. Trumbull throughout, as chairman, while Mr. Shannon was the capable chairman of the Printing Committee.

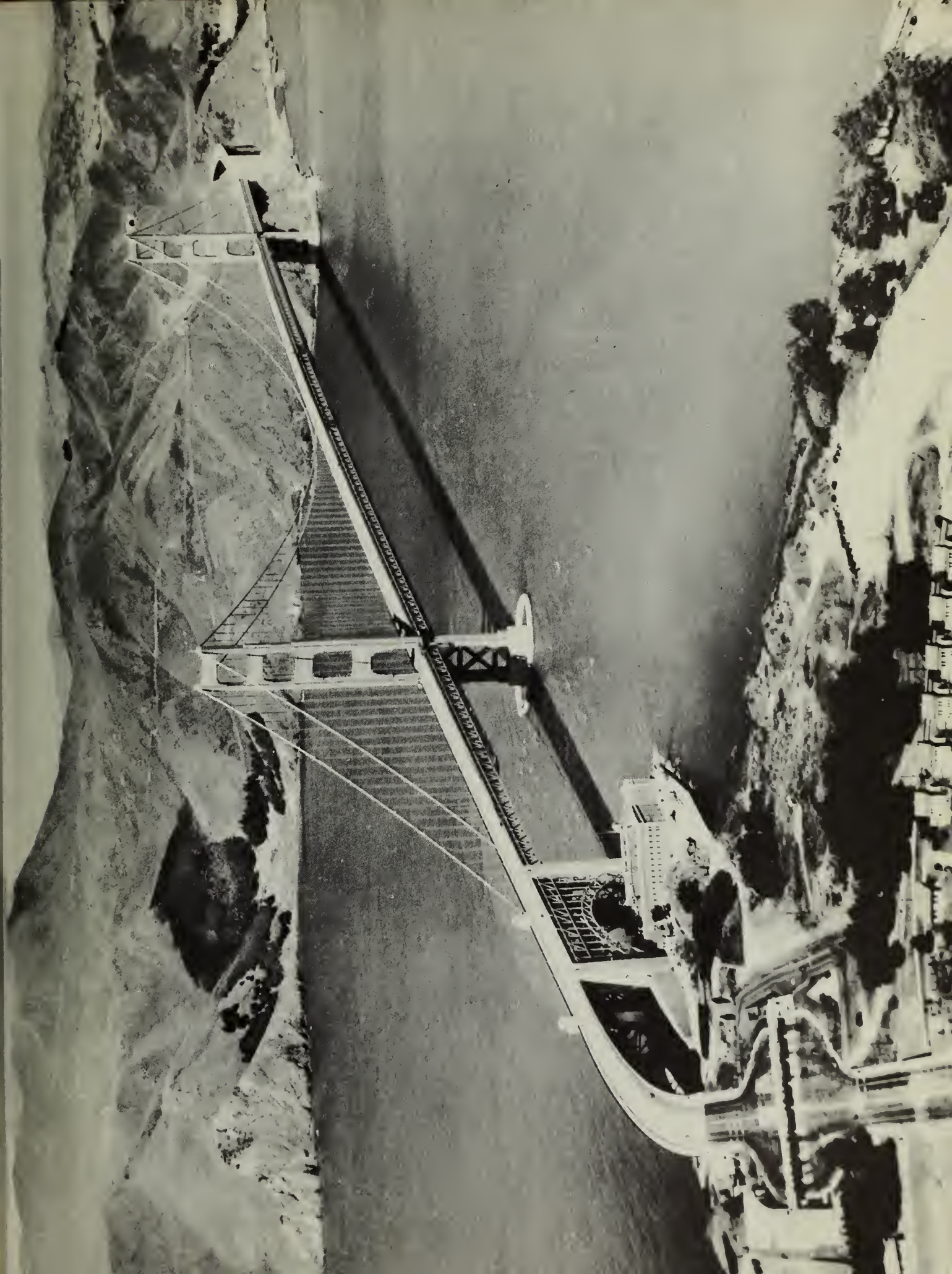
Immediately after taking office the Board was confronted with the necessity of providing funds for carrying out the purpose for which it was appointed, namely the building of the Golden Gate Bridge. The Act provides that such funds may be raised in the preliminary stages of the enterprise by taxation. Accordingly, a tax rate of 3 cents per \$100 was levied on all taxable property in the District on July 24, 1929 for preliminary expenses, and an additional 2 cents was levied in July the following year, making a total of 5 cents, yielding about \$465,000. The assessed value of all District property for taxation purposes is a little under one billion dollars.

The Chief Engineer was appointed August 15, 1929. He was selected, on the basis of a written proposal, after the Board had carefully considered proposals from nine of the leading engineers of the country. He is a bonded officer of the District.

Under the contract entered into between the District and the writer as Chief Engineer October 7, 1929, the latter was given complete charge of the engineering work of the project including architectural design, geology, traffic, inspection and supervision of construction. Messrs. O. H. Ammann and Leon S. Moisseiff of New York City, named in the writer's proposal as Consulting Engineers, were so appointed. Subsequently, the Directors added the name of Charles Derleth, Jr. of the University of California. These four men were, by contract, constituted an Advisory Engineering Board operating under the direction of the Chief Engineer as Chairman.

In November 1929, the Chief Engineer opened a local office in San Francisco in the Russ Building. A year later he moved to the Financial Center Building in offices adjacent to those of the General Manager. From these two offices he personally conducted the operations prior to the beginning of construction. In March, 1933, the District's offices were permanently located in the Hunter-Dulin Building and the Chief Engineer then moved his own offices adjacent to the District's offices in that building. This office remained throughout as the personal headquarters of the Chief Engineer and the clearing house through which all the engineering operations were conducted.

There were three additional offices maintained by the Chief Engineer; one, the home office in Chicago, where the designing, computing, detailing and the major portion of the plans were executed; two, a principal Field Office at Fort Point, serving as headquarters of the Resident Engineer, and, three, a secondary Field Office in Marin County.



As geologist, the Chief Engineer appointed Professor Andrew C. Lawson of the University of California. As traffic engineer he appointed Sydney W. Taylor, Jr. of Berkeley, and to carry out the details of the architectural design as laid down he appointed Irving F. Morrow as consulting architect. Later Allan E. Sedgwick of Los Angeles was added as associate consulting geologist. These men were all preeminent in their profession.

When the writer left Chicago to assume his duties as Chief Engineer of the District, he placed Mr. Clifford E. Paine in charge of the Chicago office, and later, in 1931, he designated Mr. Paine as his Principal Assistant Engineer on the Golden Gate Bridge and Mr. Charles H. Clarahan, Jr. and Mr. Dwight N. Wetherell of the Strauss & Paine, Inc. staff in Chicago as Assistant Engineers. From that time on, the Chicago office, with its staff of approximately forty men, was devoted almost exclusively to the Golden Gate Bridge work. Most of the development of the design and the details and most of the plans were executed there.

Later on, as the work in the field progressed, the Chief Engineer arranged for a division of Mr. Paine's time between San Francisco and Chicago and during the last three years most of Mr. Paine's time has been devoted to assisting the Chief Engineer in the San Francisco office, where, under his direction, he supervised the work in the field and shops to insure its execution in accordance with the intent of the design.

On February 15, 1933 Mr. Russell G. Cone, whom the Chief Engineer had earlier appointed as Resident Engineer, entered upon his duties in the field at San Francisco, with Mr. Theodore M. Kuss as his assistant. At the same time Mr. W. J. Evans was placed in charge of the surveying staff in the field. These men with their staff were housed in the field offices at the site.

Inspection, being a highly important function of the Chief Engineer, was organized with great care. After thorough consideration Mr. Herbert J. Baker was selected to organize and head the District's eastern inspection staff with headquarters in New York City. This staff inspected the work performed in the eastern mills and shops. Inspection in the western mills and shops was placed with Smith, Emery and Company of San Francisco.

The total engineering force, including designers, detailers, surveyors and inspectors numbered approximately one hundred men. This staff, operating under the direction of the Chief Engineer, handled all of the work on the project from its inception to its conclusion, with one exception, namely that on February 1, 1933 the Board of Directors by resolution transferred the planning of the Sausalito Lateral work to the California State Highway Commission. The work was executed as a W.P.A. project sponsored by the District. Certain parts of the Military replacement program were similarly handled. The Waldo Point Approach and the Funston Avenue roads, in accordance with the original program, are a part of the California State Highway Commission's contribution to the project and the design and construction of these roads as built and to be built are in their hands.

CHIEF ENGINEER'S REPORT OF 1930

The first meeting of the Board of Engineers was held in San Francisco during August 1929. This meeting resulted in the adoption of design specifications and definite methods of procedure, including among other things the decision to carry out immediately the necessary exploratory drilling and soil tests in connection with the foundations and anchorages. In the interval which had elapsed any advantages possessed by the cantilever-suspension type bridge had practically disappeared and on recommendation of the Chief Engineer, the cantilever-suspension type was abandoned in favor of the simple suspension type.

The following month preliminary surveys were started in connection with the bridge approaches in the Presidio and Fort Baker military reservations. Conferences followed with civic organizations and improvement clubs, with the military authorities, the City and the State for the purpose of ascertaining their views respecting bridge outlets and road connections. Negotiations were concluded with the Highway Commission whereby they would undertake the construction of the Waldo Point and Funston Avenue roads as a part of the State Highway system. And in October the writer accompanied President Filmer to Washington, D. C., where during a conference with the Secretary of War the latter stated that the provisional permit issued in December, 1924, was regarded as an agreement, but that it would be desirable later to hold another hearing for consideration of a final permit.

Bids were invited for borings at the site of the south pier a short time later and the E. J. Longyear Exploration Company was awarded the contract. Meanwhile Geologist Andrew C. Lawson had begun his geological studies, and preparations were made for carrying out the soil pressure test behind Old Fort Point, the results of which the reader may have noted in his reading of the section on Geology.

The second session of the Engineering Board was held in San Francisco in February, 1930, the General Manager, Traffic Engineer, and Geologist Lawson attending. Results of the borings, soil pressure test and other foundation data gathered since the Board's previous session were examined and it was concluded that foundation conditions throughout were entirely satisfactory. It was at this session that the Board approved the Chief Engineer's recommendation that the span be increased to 4200 feet in order to permit the location of the north pier on a projecting ledge at the Marin shoreline. This would minimize under-water work. The Chief Engineer presented his outline studies of the general design, the architectural treatment and the structural approaches—the latter based on non-interference with Old Fort Point. The main dimensions of the structure were fixed in accordance with this design, remaining substantially the same as those of the original design, except that the distance center to center of main trusses, on Mr. Moisseiff's suggestion, was increased from 80 feet to 90 feet. The original type of floor system was retained, but the surface tracks provided in the original design were eliminated. It was felt that the era of surface cars as a means of rapid mass



transportation had passed. This provided six clear 10-foot traffic lanes and two 10-foot sidewalks. Tentative design specifications were agreed upon, and the Traffic Engineer presented his preliminary traffic report.

The hearing for the final War Department permit was set for June 30, 1930, in the Board of Supervisors' Chambers, San Francisco. Meanwhile the organized opposition had not been slumbering. Here was another ready-made opportunity, and this time the shipping interests assumed the role of principal spearhead. The hearing was conducted by a Special Army Board comprising three high-ranking officers of the Engineering Corps appointed by the Secretary of War. The District's case, conducted by the Chief Engineer, was supported by a battery of experts, each a distinguished specialist in his own particular line. The protestants did not contest the horizontal clearance. They merely questioned the adequacy of the vertical clearance, and in addition, pointed out that enemy bombing or gun fire, by destroying a bridge located at the very entrance of the harbor, would without doubt most effectually bottle up the harbor. They contended further that the vertical clearance in a bridge spanning the entrance of a harbor like San Francisco's should be unlimited, in other words, no bridge at all should be permitted across the Golden Gate.

Upon conclusion of the hearing the Special Board, after due deliberation forwarded its report to General Lytle Brown, the then Chief of Engineers of the U. S. Engineer Corps. Shortly thereafter the writer accompanied by Attorney Harlan, went to Washington where, after an extended hearing, and with the help of California's Senator Shortridge and Congressman Welch, the writer's position was sustained, and on August 11, 1930, the War Department issued its permit on the basis of a 4200-foot span and a vertical clearance of 220 feet at mid-span and 210 feet at the towers. General Brown stated that he was opposed to granting the permit, but that the provisional permit of 1924 left him no alternative.

These preliminaries paved the way for the preparation of the Chief Engineer's official report to the Directors which, according to the program laid down, was to be submitted to the electors of the District as the official plans and estimate upon which they were to vote. The report briefly described the project, summarized the data gathered since the writer's appointment as Chief Engineer and included general plans, architectural sketches, layout of approach roads, terminals and Toll Plaza, estimates of cost and forecasts of traffic and revenue. In short this report covered the entire project. It was completed August 27, 1930 and issued in three volumes—Volume I being the printed report proper, with Synopsis; Volume II, the printed report on traffic matters, and Volume III, the drawings.

In submitting this report the writer informed the Directors of the District that he had, in many conferences with the responsible State Highway and City officials and officers of the Ninth Corps Area stationed at the Presidio, reached general agreements on certain military replacements and the alignment of the approach roads and the points at which traffic should be delivered; and that he had been successful, as well, in obtain-

ing a firm commitment from the California Highway Commission to the District that the State would build the bridge's main north-south approaches on both sides of the Golden Gate.

The report showed an estimated construction cost of \$27,165,000. The Board of Directors of the District concluded that the proceeds of a \$35,000,000 bond issue would be ample to meet this construction cost plus a liberal allowance for administration, engineering and financial costs. It was accordingly decided that the project should be presented to the voters of the District in the form and manner as described in the Chief Engineer's official report of August 27, 1930.

THE BOND ELECTION

While these steps were being taken by the District, the opposition had organized a body known as the Citizens' Committee against the Golden Gate Bridge bonds, numbering among its members many businessmen, professional men and taxpayers who were honestly convinced that the bridge was physically and financially impossible. This committee took space in a downtown office building, organized a considerable staff of paid employees, and set about with the aid of speakers—volunteer and hired—with broadcasts, paid advertising and printed matter, with house-to-house canvassers, to put on a determined campaign against the bonds.

Debates were staged by the opposition at various places. The radio was widely used. Women's organizations and others were beseeched to stop what was termed an outrage and a wildcat scheme. Directors and officers were subjected to a constant campaign designed to shake their confidence and break down their resistance.

The committee's propagandists claimed that a bridge across the Golden Gate would mar the natural beauty of San Francisco's world-famed harbor entrance, that it would destroy Sausalito's splendid isolation, that that city would be overrun by week-end picnickers, and that an enemy fleet with well-directed gunfire from distant points offshore could demolish the bridge and bottle up our fleet.

The earthquake hazard was enlarged upon and the false information which had been circulated regarding the south pier site was expanded. Thirteen local engineers signed a roundrobin asking that the bond issue be defeated. This roundrobin and other alarming statements were given wide circulation. A banker of standing characterized the bridge as an economic crime.

The picture painted of Marin was that of a sparsely settled territory which would not support sufficient traffic to permit a toll bridge to meet its financial obligations. Constantly reiterated throughout the whole campaign was the charge that the cost of the project was grossly underestimated. This thought was so deeply bedded in the minds of the public that it persisted almost up until the time the bridge was opened to traffic. Articles in the local and eastern press spread the story throughout the nation until the public generally came to believe that the bridge could not and would not be built.



The writer in the beginning of this campaign was authorized by the Directors of the District to organize a Bureau of Information in order that the opposition's uninformed statements and misleading propaganda and fear campaign might be counteracted and the voting public given the true facts. This bureau was placed under the direction of Charles W. Duncan, aided by two technical experts assigned to him from the Chief Engineer's staff. Mr. Duncan selected Miner Chipman, who, as head of the newly created organization's Speakers' Bureau (in which the Junior Chamber of Commerce cooperated) performed notable service in addresses to the civic and improvement clubs all over the district and in the broadcasting rooms of the radio studios.

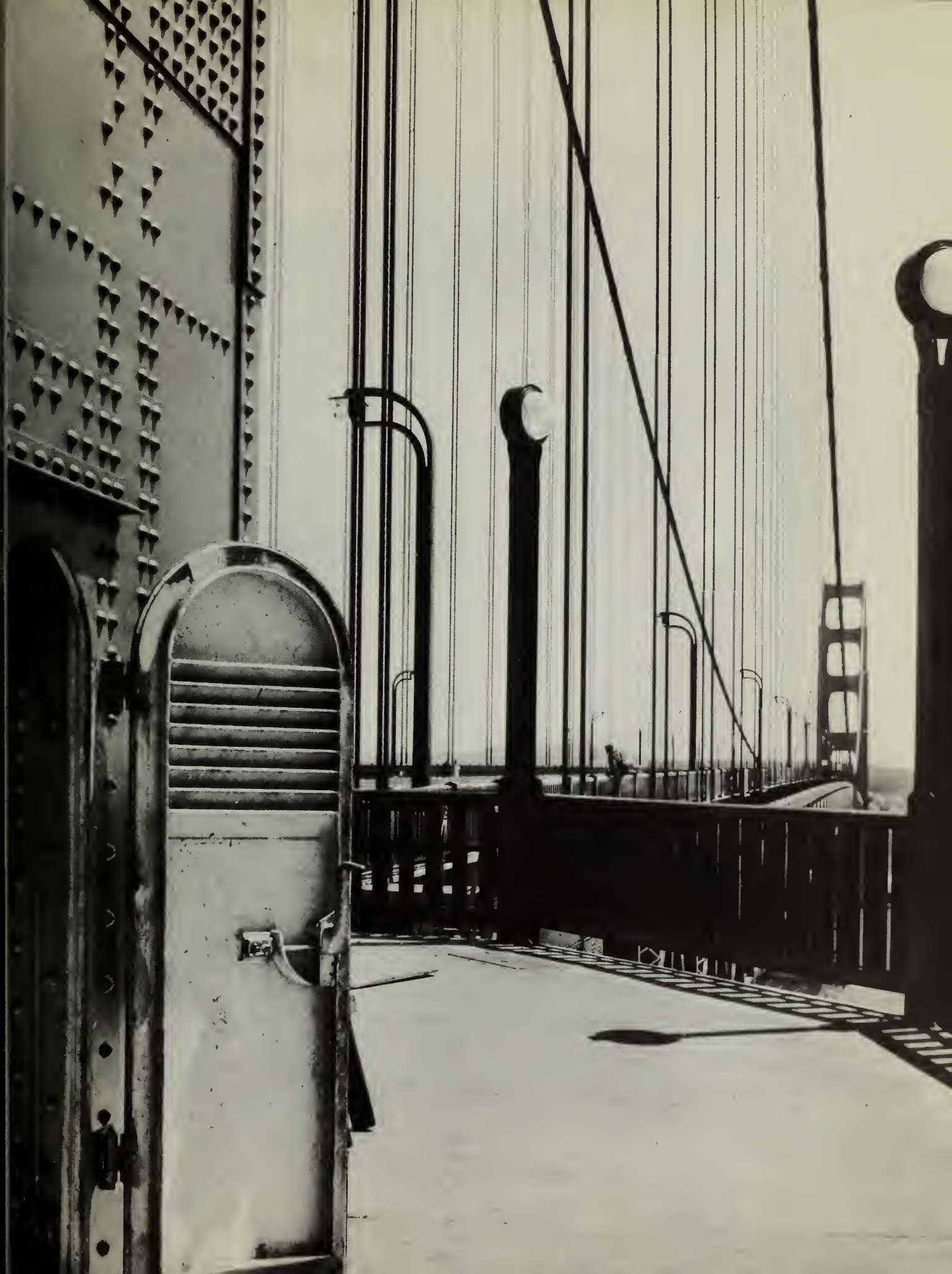
This Bureau of Information, ably supported by the bridge Directors, all of the officials and consultants, and with some exceptions the press, was successful in securing endorsement of the project by one hundred separate civic organizations, and as a result of its intelligently directed labors the electors of the District, on November 4, 1930, by a majority well over the two thirds required by law, voted approval of the issuance of \$35,000,000 face value of Golden Gate Bridge and Highway District bonds.

BIDDING PLANS AND BIDS

With the District's bonds approved by the unprecedented majority of 145,057 to 46,954 the way now seemed cleared to start the project without further delay, and the Chief Engineer was so directed. Accordingly, arrangements were made with the Coast and Geodetic Survey for a triangulation survey, and conferences were held with the State Highway Engineer in connection with the State's program for building the Waldo Point and Funston Avenue approach roads and with the military authorities in determining the rights of way for these roads within the Fort Baker and Presidio reservations.

We have noted the opposition's insistence throughout the bond campaign that the Chief Engineer had grossly underestimated the project's cost. If, then, the District's Directors could be induced to pledge themselves that construction would not be started unless the total of the contractors' bids came within the amount of the bond issue, the project by this means might effectually be killed. The Directors, toward the end of the bond campaign, with entire confidence in the estimates, had so pledged themselves. But while they were influenced in so doing partly by the opposition's attack, they nevertheless felt that the electors of the District were entitled to definite assurance that \$35,000,000, and only that sum, would be the total amount of the financing. We shall presently see that the total of the contractors' bids came well within this amount, leaving a comfortable margin for interest during construction and all other costs besides.

Pursuant to their pledge, therefore, the Directors ordered the Chief Engineer to prepare plans and specifications covering all units of the work so that lump sum bids could be received and the construction cost of the project ascertained in advance of the awarding of any contracts. Such procedure, of course, is contrary to standard practice on major jobs, where the starting of certain units of construction must, in the nature of



things, be postponed until other units have been finished. Nevertheless the situation in this instance required the adoption of this program.

To the end that construction be gotten under way as early as possible, it was decided that the Bidding Plans should be so drawn as to enable bidders to accurately prepare their lump sum bids, but at the same time allow the greatest possible flexibility in the subsequent development of construction plans for those portions of the work which would not be started until later. It was realized also that proper provisions should be made in the proposals and contracts, whereby adjustments in the contractors' compensation because of changes in plans could be on a basis agreed upon in the contracts, which was done.

The work was divided into ten principal contracts including for completeness, provisional plans covering the Toll Plaza, although this unit was among the last to be built. As to approaches, the plans included (1) a lateral to Sausalito's south limits leading from a braided connection with the Waldo road, and (2) the so-called Presidio Approach Road terminating at the end of Marina Boulevard in San Francisco. The remaining outlets, unfortunately, but through no fault of the District, were not contracted for completion synchronously with the completion of the bridge, with the result that when the bridge was completed, it lacked adequate and properly placed feeders to its main approaches.

Bids were advertised for as returnable in July, 1931. The following tabulation summarizes by unit numbers and character of work covered, the low bids received on that date:

I-A	Steel Superstructure.....	\$10,494,000.00
I-B	Steel Cables, Suspenders & Accessories.....	6,255,767.65
II	San Francisco Pier and Fender & Marin Pier.....	2,260,000.00
III	Anchorage & Piers of Approach Spans.....	1,645,841.28
IV	Steel Superstructure, San Francisco and Marin Approaches.....	996,000.00
V	Presidio Approach Road.....	966,180.00
VI	Sausalito Approach Road.....	67,586.00
VII	Paving of Main Span, Side & Approach Spans.....	345,000.00
VIII	Electrical Work.....	133,495.00
IX	Toll Houses & Service Buildings.....	71,430.00
X	Cement (Estimated 500,000 bbls.).....	1,220,000.00
Total.....		\$24,455,299.93

In addition to the above, there were certain minor miscellaneous items which could of necessity only be covered by estimates. The Chief Engineer's estimate of construction cost, as contained in his Report of 1930 totaled \$27,165,000. In the light of the bona-fide bids above noted, it was indisputably evident that the total cost would not exceed the Chief Engineer's estimate.

Since the project cost, including interest during the construction period and all other costs and allowances, was thus established at a figure comfortably under the

amount of the bonds, the pledge of the Directors had been fulfilled. However, the District was still not in position to award the contracts and begin the work because of the new litigation referred to in the following section. Accordingly, provisional agreements were entered into with the bidders on the principal contracts in the expectation that the District might retain the benefits of the bids received. On certain other contracts, such as Contract VI, bids were rejected because of the decision to assign the planning of this lateral to the California State Highway Commission as already stated. The bid on the paving was also deferred.

SECOND STAGE OF LITIGATION

As the reader has already seen, there was extended litigation in connection with the formation of the District and it was assumed that this was all that was involved, but it developed before the work could be proceeded with that it was necessary that California's Supreme Court first clarify certain constitutional questions having to do with the District's power to tax. These questions had been raised by investment bankers—prospective bidders for the District's bonds—and one of the contractors. Clarification was essential before the bonds could be marketed.

In view of its urgency the Court heard the matter without undue delay, and, although the opposition through counsel intervened in the proceeding, the Court's favorable decision was soon rendered, only one Justice dissenting.

With the favorable decision of the Supreme Court behind them, officials of the District hoped that the project had taken its last legal hurdle and that they could now sell their bonds, award contracts, and proceed uninterruptedly with the business of building the bridge. But new litigation developed, this time in the United States District Court, the plaintiffs being the Del Norte Company and the Garland Company,—two tax-paying corporations from the North.

The District thus found itself defendant in an action which might conceivably continue litigation for several years. Whether the District in these circumstances could be held together that long was by no means certain. But the Directors, having received the people's mandate at the bond election to build the bridge, engaged special counsel to carry out a vigorous defense.

In the spring of 1932 with Orrick, Palmer and Dahlquist as associate counsel, the District won in the trial court. At that time men walked the streets looking for employment and the equipment of steel mills and cement plants were depreciating in idleness. Nevertheless, the opposition decided to appeal, and carry the case to the Supreme Court if necessary. But an aroused public had meantime become more and more vocal, contending that the electors of the District had voted the bonds by a huge majority and that the opposition had had its day in court. To the cumulative force of this pressure the opposition at last gave way and instructed counsel to withdraw the appeal, and accordingly this litigation was terminated in July, 1932.



But just as steps were being taken to proceed with the letting of the contracts, a new source of delay arose. The Bankamerica Company, heading a syndicate to finance the District, had bid 92.3 for the first offering of bonds, making the effective rate of interest 5.25 per cent. In the opinion of the District's legal advisors in New York, which, by the way, conflicted with the local legal opinion, this rate exceeded the permissible rate, since the Directors had asked the electors of the District for approval of 5 per cent bonds. Nevertheless and although this was nothing more than a technicality, it was decided that only a court trial could remove the doubt as to the legality of the bonds. The District's funds by that time had become exhausted and the District, although victorious in every legal encounter, feared to levy a new tax to carry on in the face of this new and unexpected obstacle. Furthermore, litigation would consume valuable time and indefinitely postpone the starting of construction.

Realizing the seriousness of this situation, the Chief Engineer with a strong committee of Bridge Directors and officials presented the District's problem to A. P. Giannini, chairman of the Board of the Bank of America. Mr. Giannini pledged his bank's support, and, as a result and through the splendid work of Will F. Moorish (the bank's president at that time) and local bond houses, a new bond syndicate headed by the Bank of America was formed. Notwithstanding an unfavorable bond market, this syndicate agreed, pending a test of the legality of the interest rate, to purchase \$3,000,000 of the District's bonds at 96.23, making the effective rate of interest 5 per cent, in accordance with the interpretation of the New York counsel. In addition, the syndicate, pending determination of this technical question that had been raised, advanced \$184,600 against \$200,000 of the District's unvalidated bonds, as suggested by the Chief Engineer, as a result of which the District was provided with the funds it so urgently needed for current expenses. The consummation of the project was thus assured and the District was in a position to proceed with construction.

AWARD OF CONTRACTS

Due to the unexpected prolongation of litigation, so much time had elapsed since the receipt of bids in July, 1931, that the low bidders had been released from their proposals with the single exception of Contract I-A. In the case of this contract, which covered the Steel Superstructure of the Main Span, the proposal was continued in effect, but it was decided to call for new bids on other units of the work. It was also decided to have these new bids include the cement as a part of the concrete work instead of calling for separate bids as had been done in the former bidding.

It was also decided to omit from the bids the Toll Terminal and the final painting, the former because the Toll Terminal grant of the government gave an inadequate area and the new area contemplated would of necessity invalidate the original layout, and the latter because it was a minor item, the performance of which was at least four years off.

The new bids, received October 14, 1932, therefore, together with the former bid on Contract I-A, covered nine contracts—I-A to VIII inclusive—and were as follows:

I-A	Steel Superstructure (McClintic-Marshall Corporation)	\$10,494,000
I-B	Steel Cables, Suspenders & Accessories (John A. Roebling's Sons Company)	5,855,000
II	San Francisco Pier and Fender & Marin Pier (Pacific Bridge Company)	2,935,000
III	Anchorage & Piers of Approach Spans (Barrett & Hilp)	1,859,855
IV	Steel Superstructure, San Francisco and Marin Approaches (J. H. Pomeroy & Co., Inc., and Raymond Concrete Pile Company)	934,800
V	Presidio Approach Road (Eaton & Smith)	996,000
VI	Sausalito Approach Road	59,780
VII	Paving of Main Spans (Barrett & Hilp and Pacific Bridge Company) ..	555,000
VIII	Electrical Work (Alta Electric & Mechanical Company, Inc.)	154,470
Total		\$23,843,905

In November, 1932, contracts were accordingly awarded on the basis of these bids for I-B, II, III, IV and VIII and tentatively for V. Contract VI (the Sausalito Lateral) was not awarded and was later built as a W.P.A. project, as already stated.

BUILDING THE STRUCTURE

Construction was officially begun January 5, 1933. In commemoration of this event officials of the District on February 26 staged a ground-breaking ceremony at Crissy Field, the like of which for pageantry and enthusiastic support of the citizenry had never before been witnessed in the bay region. Director William P. Stanton was chairman of the committee in charge, and the writer's office was the clearing house through which the committee functioned. Participation of the United States fleet had been secured; people flocked to San Francisco from distant points, and the mayor officially proclaimed the day a holiday. Work on the San Francisco and Marin anchorages was started close on the heels of the ground-breaking ceremony.

We shall note here only the principal events of interest in connection with the project's building and leave for discussion in subsequent chapters such other matters as the details of its construction and design and the standards set up for the testing and inspection of materials. But first we should record that, while the Chief Engineer was in full charge of all work relating to the design and building of the project, the Directors had entered into an agreement with the California State Highway Commission for the planning of the Sausalito Lateral in order to facilitate the tie-in of this work with the Waldo Point Road and its construction in accordance with State Highway standards.

The building of this lateral, a two-lane road, was begun in April, 1936, by the State Division of Highways and carried out as a W.P.A. project sponsored by the District.

Certain of the military replacements and improvements required of the District in the Presidio and Fort Baker reservations were similarly handled—as W.P.A. projects. The Waldo Point and Funston Avenue approaches, as we have seen, were to be financed and built by the State. The former was completed as a four-lane road and made ready for traffic the day before the bridge was opened. The negotiations for the latter, at this writing, however, are still in process.

No especial problems had to be met in the building of the Marin pier. It was necessary only to enclose the area on three sides by a cofferdam of steel sheet piling buttressed within by rock-filled timber cribbing, maintain the pumps in continuous operation, and proceed. But the building of the San Francisco pier was quite a different story.

We have previously called attention to the exposed storm-swept location of the bridge site—the heavy ground swells, prevailing high-velocity cross winds and tidal currents running upwards of seven knots. The San Francisco pier, 1125 feet offshore, lies virtually in the open sea and is wholly unprotected from the elements. The contractor's first move toward the construction of this unit was to build an access trestle 22 feet wide and 1100 feet long from the San Francisco shore to the pier site, bombing each bent shoe into the solid rock of the ocean's floor as construction progressed.

The access trestle had hardly been completed when a vessel off its course in a thick fog crashed through it, not without considerable damage to itself, and the resulting breach had no sooner been repaired when a section of the structure 800 feet long was carried away in a storm. The Chief Engineer then ordered the trestle raised five feet and securely guyed at intervals to the bedrock by anchored, steel cables. Thus reinforced, it stood without further mishap until the end of the job, serving effectively over the four year construction period as a means of access for the contractors not only for the San Francisco pier but for the steel superstructure as well.

The fender, an original conception designed to play the dual role of a cofferdam within which the excavation and building of the pier could be carried on and a permanent fender to protect the pier from the impact of passing vessels, will be described in detail in following sections. The pier proper was to be built within this fender by means of a pneumatic caisson. The contractor had planned to float the caisson into position through an opening left in the east end of the fender for that purpose, the opening to be closed as soon as possible after the caisson was inside. The caisson was built at the Oakland yard of the Moore

Mr. Filmer, Mayor Rossi, and Mr. Strauss at the ground-breaking ceremony on February 26, 1933



Dry Dock Company and successfully floated into place, but by nightfall of that same day, under the influence of unexpected heavy swells of unknown origin, it became a battering ram which threatened to demolish the fender wall. At a midnight conference the Chief Engineer gave the contractor permission to remove the caisson, a hazardous job, which was, however, successfully accomplished.

The fender ring was then closed, the inside filled with tremie concrete up to elevation —35; using the fender as a cofferdam, the water was pumped out and the building of the upper portion of the pier was completed in the dry. This was not done without hazard, as was evidenced in December, 1934, when the prow of a great ocean freighter suddenly loomed out of the fog with but a few feet to spare. Nevertheless after 20 months' patient labor, including 11 months' delay, the pier was completed.

Subsequent to the bond election, the opponents' criticism of the south pier foundation persisted and in order to quiet this the Board of Directors authorized a further and separate investigation of the foundation conditions existing at the pier site and upon completion of this investigation accepted the conclusions of its consulting geologists Andrew C. Lawson and Allan E. Sedgwick, which confirmed that the safety of the foundation was beyond question. Nevertheless, after the pier was well along towards completion a new attack was made by a local and meddlesome geologist and this was given such widespread publicity that the whole subject was again given a further hearing by the Board of Directors, its Engineering Board and consulting geologists, with the result that this further criticism was found to be equally fallacious. This finally ended the insistent campaign to impeach the safety of the San Francisco pier.

Consideration had been given by the Board of Engineers to the use of high-silica cement in the construction of the San Francisco pier and fender. Their investigation convinced them that this cement would produce concrete having superior qualities in sea water exposure and they therefore recommended its use. So far as the writer knows this is the first major work to employ such cement.

During the progress of the work on the south pier, the north pier and tower had been completed and the work on the two anchorages and pylon groups had proceeded uninterruptedly. Some time after work on the Presidio Approach road had begun, objection was raised to its terminus at Marina Boulevard, on the ground that the bridge traffic would increase the congestion on the boulevard intolerably. The District was therefore asked to abandon this terminus and divert all the traffic to Lombard Street. After protracted negotiation between the City, the War Department, the Park Commission and the District, an agreement was reached on a compromise solution proposed by the Chief Engineer, whereby the Marina Terminus was retained as planned and a secondary outlet, for truck traffic principally, was carried behind the Palace of Fine Arts joining with the Presidio Approach by means of a braided connection and connecting with a new diagonal street now called Richardson Avenue and thence with Lombard Street. This outlet was subsequently built as a W.P.A. project sponsored by the city.

The building of the two Presidio viaducts and the Marin approaches involved the demolition and reconstruction of more or less elaborate systems of sometimes outmoded military structures and works, under the supervision of the army engineers. The work performed on this phase of the project includes among other things the construction of several fire control stations, a \$125,000 modern powder magazine, a rifle range, extensive machine and other shops and gas stations, drainage and sewerage systems, living quarters and roads.

The District, by the terms of its permit to use and occupy certain reservation property, agreed to replace and make good the damage to military structures and facilities wherever there was any interference. This was an equitable and proper return for the valuable concessions given the District in rights of way, but due to governmental requirements it was a most exacting and trying phase of the work, and for the same reason its cost was an item which constantly grew larger.

The Toll Terminal site, located 170 feet south of the south abutment of the bridge, had been agreed to by the Army as originally determined in the preliminary lay-out plans, but in making the grant, the War Department had limited the area to 200 feet by 300 feet. The Chief Engineer from the outset protested the insufficiency of this area and finally prevailed upon the War Department to reconsider the matter. Accordingly in addition to a liaison officer appointed by the War Department to deal with the District, a local Board of Army Engineers was set up for the specific purpose of reviewing this matter as well as the reconstruction of various batteries, the planning of

Work progressed simultaneously on the Anchorages, Pylons, Marin Tower and San Francisco Pier



the powder magazine and other matters relating to the work in the Presidio and Fort Baker. This Board held formal hearings and after many months of negotiations a satisfactory agreement was reached on all major matters involved, including the expansion of the Toll Terminal area from 200 by 300 feet to 350 by 500 feet. Although the latter dimensions were somewhat less than those urged by the Chief Engineer, it was decided to accept them and a revised permit was issued on this basis.

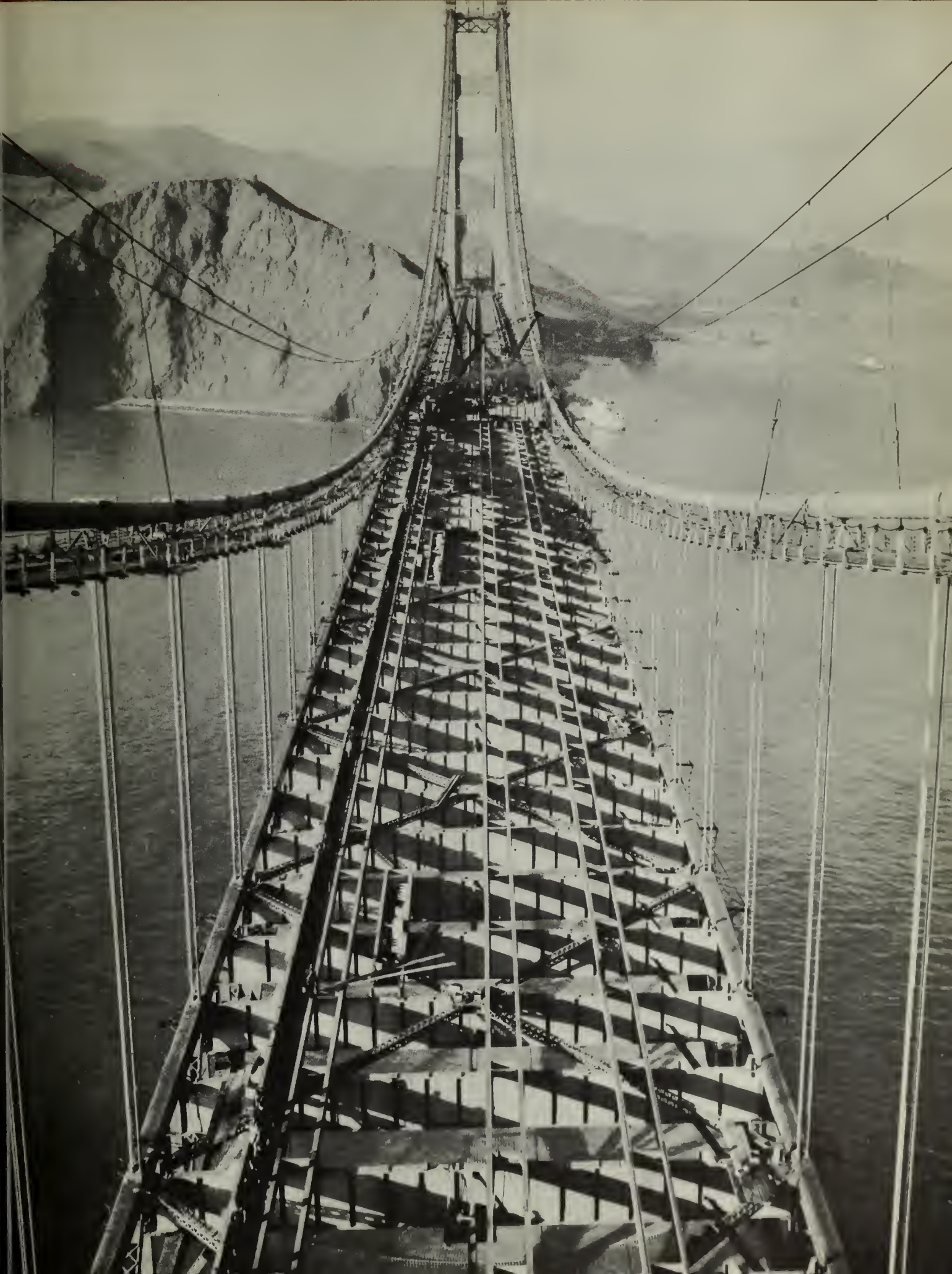
It was necessary to divert Lincoln Boulevard just south of the Toll Plaza and reconstruct approximately a quarter-mile length thereof, depress it and construct an over-pass across it to connect the Presidio Approach road with the Toll Terminal.

CONSTRUCTION PROCEEDS: The completion of the south pier was followed in quick succession by the erection of the south tower, the installation of the cable catwalks and the spinning of the cables, the placing of the suspended structure, and the paving of the bridge approaches and floor—and the contractors for these units of construction in most cases established new records for speed.

A highlight of the work was the concern given, throughout the construction period, to the protection of the workmen. Most conspicuous among these precautions was the safety net, suspended beneath the floor system from end to end. This was a costly expedient, and because it was new and unusual it met with resistance. But the Chief Engineer's plea to the Board of Directors that if one life were saved the expenditure was justified, prevailed and the net was installed and was the means of saving nineteen lives.

As a result of these measures there had been but a single fatality up to February 17, 1937, since the beginning of the work, establishing a new all-time record in a field of activity where one man killed per each million dollars expended has been axiomatic. Notwithstanding these precautions, the unexpected happened when on February 17, 1937, one of the stripping scaffolds installed by the paving contractor fell, carrying with it twelve men and 2100 feet of the safety net. This deplorable accident cost the lives of ten men. The work was seriously delayed. Order was ultimately restored; new stripping scaffolds were put in operation, but this time only after they had been checked and approved by the Chief Engineer; the destroyed net was replaced and the work proceeded without further fatalities. The record of life lost on this structure, therefore, stands at eleven, which is unusually low.

The last unit of construction to be completed was the Toll Terminal, located in the Presidio just south of the San Francisco abutment. The preferred plans called for a reinforced concrete unit of imposing appearance in two wings separated by steel and glass toll booths surmounted by a concrete canopy which served as a connecting walk between the two wings. This design was too costly, and after two revisions the present simpler layout was adopted, and although belatedly begun, was nevertheless finished in time for the bridge opening. While not in strict conformance with the architectural requirements of the structure proper, the Toll Terminal is a compact and efficient unit



which contains, in addition to administrative offices, quarters for the operating personnel and the California Highway patrol, and a complete maintenance section, including machine shops, garages, and a power house. The latter is a modern power station in every respect, serving the entire project, and will be described hereinafter.

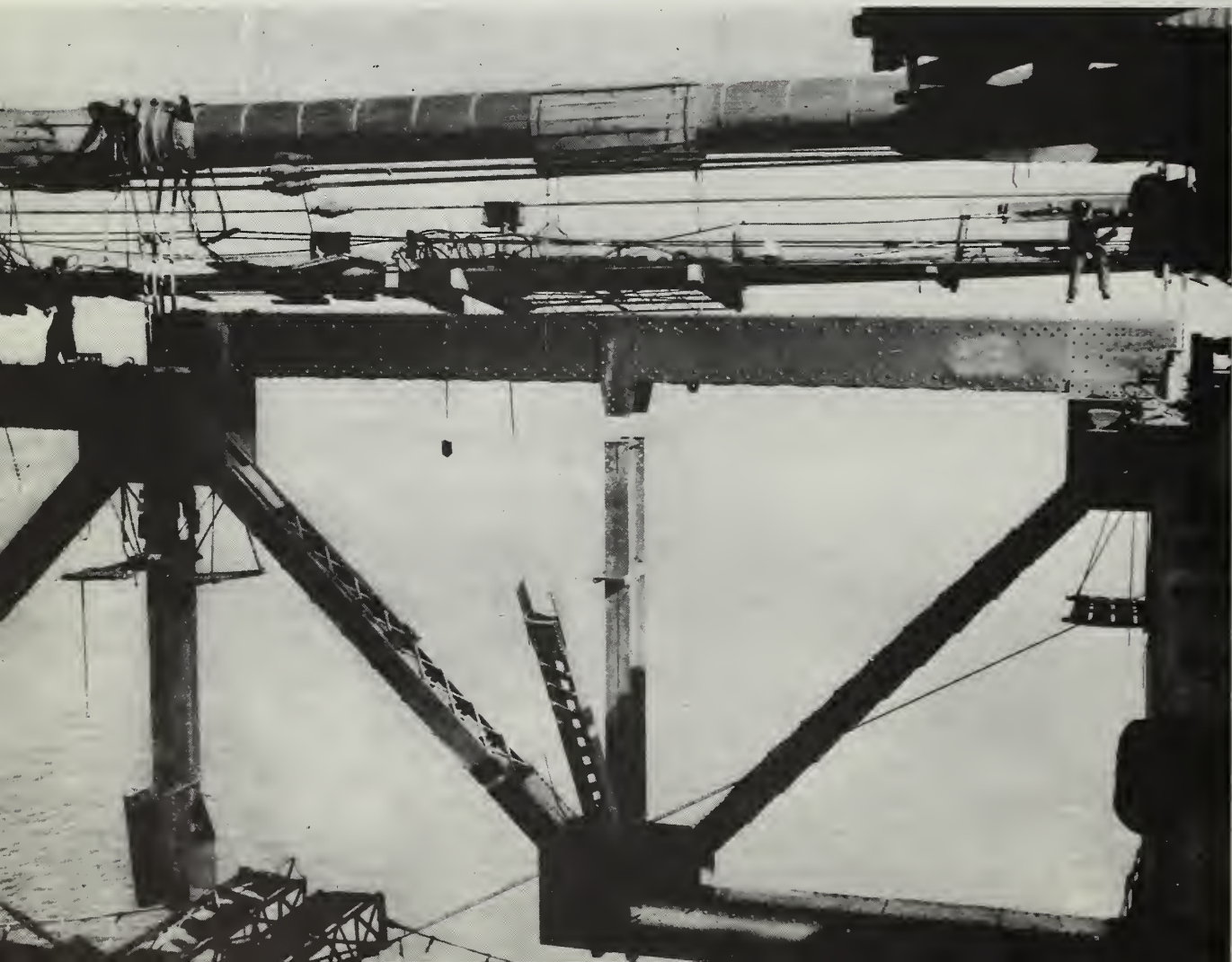
It was decided after careful consideration to equip the bridge throughout with sodium vapor lights. Originally these lights were to be limited to the Presidio Approach and the bridge proper, but later they were extended to the Sausalito Lateral and it is the expectation that ultimately the Waldo Point road will be similarly lighted.

Located on the fender of the south pier is a modernistic concrete shaft, 35 feet high, on which is mounted an aid to navigation light maintained by the District under the requirements of the United States Lighthouse Department.

Electrically-powered elevators operate in the east leg of both towers for purposes of inspection and maintenance.

The two 10-foot sidewalks are limited to the bridge proper. On the Presidio Approach there is a 3 1/2-foot sidewalk on the east side only. There are no sidewalks on the Waldo Point road and the Sausalito Lateral, but a steel underpass at the north pylon connects the two sidewalks of the bridge and permits foot passengers to go from one side to the other without crossing the roadway. Provision has been made by the United States government for parking areas near the Toll Terminal and also at the Marin terminus near the junction of the Sausalito Lateral with the Waldo Point road, which gives

The closing top chord member is being placed



opportunity for pedestrians to park their cars and walk across the span. No provision has as yet been made for commuter traffic or rapid transit, mass transportation being served by existing facilities.

COMPLETION: The Golden Gate Bridge was completed ready for traffic on May 28, 1937. However, on the preceding day the bridge was thrown open to pedestrians only. It was a gala occasion marking the beginning of the Golden Gate Bridge Fiesta and from morning until night the bridge was packed with 200,000 joyful celebrants.

On the following day, May 28, the Chief Engineer in a brief ceremony turned the bridge over to President Filmer, who accepted it on behalf of the District. This was followed by a most impressive program at which former Director Francis V. Keesling delivered the principal address. Then came a week of gayety and carnival, including a magnificent pageant held at Crissy Field. The occasion brought to San Francisco many dignitaries from other states and from Canada and Mexico, the entire United States fleet and thousands of visitors. A striking feature of this celebration was the illumination of the bridge by means of flood-lights playing on the towers. A remarkably beautiful effect was obtained which gave the bridge the title "The Span of Gold." The celebration of the opening of the bridge was handled by a Citizens' Committee, of which Supervisor Arthur M. Brown, Jr. was chairman, Mr. Eric Cullenward, manager, and Mr. James Adam, publicity director. The results achieved were extraordinarily successful and reflect great credit on all concerned.

Directors of the District and engineers gathered to witness the closing of the Main Span on November 18, 1936







May 27, 1937, was designated "Pedestrian Day" and thousands of people enjoyed exclusive use of the new bridge on the day before it was opened to vehicular traffic

The opening of the bridge to traffic on May 28, 1937



CONSTRUCTION COST

The following statement shows the cost of the completed structure broken down by principal items of expenditure.

STATEMENT OF CONSTRUCTION COST

Marin Pier.....	\$ 436,000
San Francisco Pier and Fender.....	2,935,000
Shore-end Pylons.....	295,000
Cable Anchorages and Housings.....	2,328,000
Main Towers—Steel.....	6,970,000
Suspended Structure—Steel.....	3,250,000
Cables, Suspenders and Accessories.....	5,910,000
Approach Piers and Structural Steel.....	1,085,000
Paving Suspended Structure and Approaches.....	800,000
Presidio Road.....	1,314,000
Sausalito Lateral Road.....	330,000
Toll Plaza.....	450,000
Toll Collection and Recording Equipment.....	72,000
Roadway Lighting and Electrical.....	270,000
Tower Elevators.....	60,000
Military Replacements and Improvements.....	575,000
Miscellaneous and Equipment.....	45,000
	\$27,125,000
Engineering and Inspection.....	2,050,000
Administrative and Preliminary Expenses less Taxes Collected.....	423,000
Financing—Net.....	4,068,000
Surplus.....	1,334,000
Authorized Bond Issue.....	\$35,000,000

In comparing the total of the construction items above, namely, \$27,125,000, with the \$27,165,000 estimated by the Chief Engineer in 1930, it should be borne in mind that the above figure includes many expenditures not originally contemplated. The actual cost of military replacements, for example, which was originally estimated by the military authorities at not to exceed \$100,000, actually ran into the substantial total of \$575,000.

Aerial view of the
Toll Plaza on
opening day



TABLE I
GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT
BOND SALES RECORD

Sale No.	Delivery Date	Par Value	Net Received	Discount	Premium	Yield	Discount	Premium	Int. Rate
1st	11-17-32	\$ 3,000,000	\$ 2,887,000.00	\$113,000.00	-----	.96233 1/3	.03766 2/3	-----	4.75
2nd	3- 1-33	3,000,000	2,887,000.00	113,000.00	-----	.96233 1/3	.03766 2/3	-----	4.75
3rd	7-25-33	3,000,000	2,800,365.00	199,635.00	-----	.933455	.066545	-----	4.75
4th	3- 7-34	1,500,000	1,399,215.00	100,785.00	-----	.93281	.06719	-----	4.75
5th	5- 1-34	2,000,000	1,920,696.85	79,303.15	-----	.9603484	.0396516	-----	4.75
6th	6-12-34	2,000,000	1,946,911.40	53,088.60	-----	.9734557	.0265443	-----	4.75
7th	1- 2-35	3,000,000	2,989,434.15	10,565.85	-----	.996478	.003522	-----	4.75
8th	2- 4-35	3,200,000	3,285,033.15	-----	\$ 85,033.15	1.0265728	-----	.0265728	4.75
9th	11-26-35	3,000,000	2,970,863.05	29,136.95	-----	.9902876	.009712	-----	3.75
10th	3-10-36	3,000,000	3,022,034.05	-----	22,034.05	1.00734468	-----	.00734468	3.75
11th	7- 1-36	5,000,000	5,151,005.70	-----	151,005.70	1.03020114	-----	.03020114	3.75
12th	6- 8-37	3,300,000	3,353,069.60	-----	53,069.60	1.0160816	-----	.0160816	3.75
Total	-----	\$35,000,000	\$34,612,627.95	\$698,514.55	\$311,142.50	-----	-----	-----	-----
Net Yield and Discount and Average Interest Rate							.988932227	.011067773	4.34



THE COMPLETED STRUCTURE

It is a truism that every great bridge project in its consummation has contributed notably to the science of structural design and the technique of the builder; and in these respects the Golden Gate Bridge has been no exception. Nevertheless, its outstanding contribution has not been to these alone, but to architectonics as well, for the structure since its completion has received notable recognition because of its majestic beauty and size.

The reader has seen that the project proper was successfully financed during the darkest days of the depression without aid of government subsidy or loan and despite the persistent opposition of influential vested interests. And he will have noted among its contributions to the arts and sciences the south pier fender, constructed first as a defense against the elements while the pier was being built, and then serving as a defense for the completed pier, and that safety measures of unusual effectiveness were instituted and carried out all through the period of construction, among them a net suspended from the floor system, already noted, which was the novel means of saving nineteen lives. Other contributions will now be recorded, and the first of these, quite appropriately, is the complete answer of the finished job itself to the fears of alarmed nature lovers, who insisted that a man-made bridge in such a setting would mar the beauty of the Golden Gate.

We have seen that the design adopted was one in which the essential beauty and elemental simplicity of a conventional suspension design was obtained, a design with symmetrical shore spans supported by the cables. To this simplicity of line was added the dignity of the well-proportioned portal-braced towers, all in accordance with the original studies. A happily-selected color scheme dominated by orange-vermilion completes the picture, blending perfectly with the changing seasonal tints of the natural setting of the bridge and the surrounding land masses, sea and sky. The effect is as highly pleasing as it is unusual in the realm of engineering structures.

The qualities of reserve power and serene dignity have been imparted to the towers by forming their two main unitary posts of built-up non-encased steel cells. The vertical lines of the tower silhouettes have been emphasized by the successive stopping-off of cells at varying heights. The vertical set-backs resulting from a rational disposition of metal give the towers extremely graceful lines. Four horizontal portal struts above the floor line—instead of the conventional system of diagonal bracing—give the towers lateral stiffness; and each strut has been enclosed by fascia plates, so fashioned as to coordinate and blend with the stepped-off lines of the towers. The strut spacing, which diminishes progressively from floor line to tower top, accentuates the height of the towers; and while this design is unique and a radical departure from traditional design, it was nevertheless accomplished without material increase in cost.

The vertical set-back motive has been carried out as well in the design of the pylons which are grouped symmetrically in pairs at the two anchorages, and the lines of the

approach structures, hand railings and sodium vapor light standards along the four-mile stretch of bridge and approach structure. Even the unsymmetrical grouping of the administration buildings at the Toll Plaza, offset to some extent by the graceful toll booths, does not materially detract from the general effect.

Old Fort Scott, dating back to the late fifties as we have seen, and still in a good state of preservation, now nestles between two pylons and beneath the 319-foot steel arch which at this point supports the bridge floor. While the old fort has no military value now, it remains nevertheless a fine example of the mason's art. Many had urged the razing of this venerable structure in order to make way for modern progress and provide an uninterrupted working area for the bridge during construction. In the writer's view it should be preserved and restored as a national monument, and that was the primary reason for the arch. Mayor Rossi, in entire sympathy with this program, has appointed a Citizens' Committee which is now planning to carry that program into effect.

The southerly pylon of the group at this point displays two bronze commemorative plaques, one on each side facing the bridge roadway where they may be read by pedestrians passing by. The plaque west of the roadway is a memorial to those heroic workers who lost their lives while construction was in progress. That on the east exhibits joint dedications, one contributed by the District itself, and the other by the Native Sons of the Golden West.

Set into the roadway on the center line of the bridge at a distance of 180 feet southward from the south abutment is a starred disk of bronze marking the center of the old fire control station at Battery Lancaster which once occupied that site. In this manner is marked the point where the first reconnaissance of the Golden Gate Bridge was initiated by the Chief Engineer. It is interesting to note that the present center line of the bridge passes through this point and that the present alignment of the approach roads, terminals and the Toll Area site of the bridge coincides with the tentative alignment established in the first reconnaissance.

FORECASTS, TRENDS AND INFLUENCES

The traffic engineer's revenue forecasts of 1930 included definite though relatively small amounts from (1) the patronage of an expected interurban rapid transit service between San Francisco and Marin, and (2) public utility concessions such as pipeline, conduits and the like, neither of which had materialized when the bridge was opened for traffic. Pedestrians, however, began at once to contribute liberally, and revenue from this source—the potentialities of which were not canvassed by the estimator—has gone beyond all expectations since the opening of the bridge. The project, however, must always rely upon the motoring public as its chief source of revenue. In bringing these historical notes to a close the effect of certain factors which have operated since 1929 to influence traffic trends will now be discussed.



HIGHWAY
PATROL OFFICE

APPROACHES: The Golden Gate Bridge is, as stated, a project conceived and justified as a toll bridge. For this reason the studies underlying the conclusion presented in the Chief Engineer's report of 1930 were limited to the examination of those factors only which would more immediately affect the financial success of the bridge, and the carrying out of the origin-destination surveys necessary for the purpose of determining approach road locations and capacities was postponed pending the approval of the bonds. Nevertheless, the traffic and revenue forecasts were very definitely predicated upon the assumption that adequate and properly placed approaches would be provided by the time the bridge was opened to traffic.

We have noted that the original plans called for three primary approach stems: one in Marin County connecting through a braided connection with the Redwood Highway at Waldo Point just beyond the city limits of Sausalito—and two in San Francisco extending through the Presidio and connecting the toll plaza (1) with Funston Avenue on the south and (2) with Marina Boulevard on the east. In addition, Sausalito local travel was to be routed over a secondary stem or lateral leading from a braided connection with the Waldo road to the town's south limits.

In compliance with regulations prescribed by the War Department, grade-separation was to be provided within the Fort Baker and Presidio reservations wherever bridge roads crossed the military network. The controlling grades contemplated were 6 per cent for the Sausalito Lateral and 4 per cent for the remaining roads. All approaches were originally planned as four-lane roads. Later developments made it desirable to change the width of the Marina Approach to six lanes and it was so built.

We have noted, too, that the District had budgeted funds for building the Marina Approach and Sausalito Lateral, and that the other feeders came within the province of the City of San Francisco and the State of California.

It is well known that two processes are involved in the building of a successful toll bridge—once the preliminary studies have made clear the project's engineering and economic feasibility. These are (1) the building of the structure proper, and (2) the linking of its portals with the established beaten tracks. At the time when the work of organizing to build the structure proper was well in hand and funds for the Marina Approach and Sausalito Lateral had been budgeted, data were still lacking on which to base the planning of effective highway feeders.

Immediately following the bond election, the Chief Engineer recommended that a comprehensive origin-destination check be authorized at an early date in order that the District's negotiators might be properly informed when they appeared in conference with officials of the City and State. But litigation intervened—litigation and the other delays recorded—and more than two years elapsed before the subject could be broached again.

This traffic check and study was authorized on February 20, 1935, and the results, with supporting data, were made available in the traffic engineer's "REPORT ON CONDITIONS AFFECTING TRAFFIC WHICH WILL USE OR MAY BE ENCOURAGED TO USE THE



GOLDEN GATE BRIDGE" transmitted to the Directors in September, 1935. Meantime the City, with State and Federal aid, was planning to improve Lombard Street as the principal bridge approach artery in San Francisco, a route dead-ending at Van Ness Avenue and even then heavily trafficked and serving as the City's main approach to the Presidio, while the Waldo Approach road was planned as a three-lane road with 5.8 per cent grade and no braided connection at Waldo junction.

The State, ultimately, agreed to build the Waldo Approach as a four-lane road and the project was later completed with its 5.8 per cent grade and without a braided connection at Waldo, in time for the opening of the bridge—nearly four miles of heavy, difficult construction and a magnificent tribute to the energy and resourcefulness of California's Division of Highways and the contractors who carried out the work. The Lombard-Lyon Street Outlet, at this writing, is nearing completion, but after that Lombard Street has still to be widened to Van Ness Avenue, a job involving property damage and the appropriation of additional funds. As yet, the City has no plans for tapping the prolific traffic resources east of the Russian-Nob Hill ridge, and the District, having completed the Sausalito Lateral as a W.P.A. project as far as the Fort Baker gate, has still to negotiate a suitable roadway into Sausalito from its southern limits.

The origin-destination survey brought to light many aspects of the proper function of the bridge as a traffic carrier not previously fully sensed. The flow diagram, Fig. 1, reproduced from the traffic report of 1935 indicates the principal and lesser sources of traffic of the bridge and shows the relative importance of each source. We have noted that two of these sources, the Sacramento Valley and the western section of San Francisco tributary to the Funston Avenue Approach, have not yet been provided with feeders. Each of these two sources, according to the flow diagram, is a potential contributor of a seventh of the total traffic of the bridge. The seventh from the Sacramento Valley will be shared, of course, with the San Francisco-Oakland Bay Bridge. But the other is the exclusive feeder of the Golden Gate Bridge, and until the Funston Avenue Approach has been completed, bridge patrons from the western section of the city will be compelled to use the Marina Approach at the expense of more than three additional miles of travel per trip, for, as we have seen, the roadways within the Presidio are permitted to have no connection with the roadways of the Bridge District.

THE TRAFFIC RESERVOIR: We have noted the instant response on the part of the motoring public in the past to every expansion of vehicular ferry service or reduction of tolls. This characteristic of San Francisco's bay crossing traffic is brought out in a striking way by the chart, Fig. 2, which was reproduced from material presented in Volume II of the Chief Engineer's Report of August 27, 1930, and subsequent records of ferry travel since 1929. The record is one of rapid, persistent growth with only temporary recessions when the depression had set in, to be followed by sharp upturns again beginning in 1933. Traffic over the Marin lines during the single month of July, 1931, for example, was higher by nearly sixty per cent than that for the entire year 1919,



FIG. 1—Traffic flow diagram

when, as we have noted, the motor vehicle began to be a factor to be reckoned with as a means of transportation.

The forecasts of 1930 were predicated upon the assumption that the bridge would be opened for traffic January 1, 1936, and that the District would pay five per cent interest on its bonds. A definite program of bond redemptions was worked out, and the traffic and revenue estimates were based on vehicular toll levels beginning with 84.3 cents per average unit and diminishing each five-year period thereafter as the chart shows. Due to litigation and the other unavoidable delays we have noted, the structure was not completed and opened for vehicular traffic until May 28, 1937. Meanwhile the District had been able to place its bonds at the average rate of 4.34 per cent, and the Directors, adhering to the original program of redemptions, had set the bridge's vehicular toll level at approximately 50.9 cents per average unit, since the San Francisco-Oakland Bay Bridge, opened for traffic November 12, 1936, had already reduced its tolls approximately to that level the following February.

For the sake of completeness, the chart shows, also, the indicated traffic and revenue for the first operating year of the bridge, together with the total for bond interest and estimated operating expenses. Bond redemption, under the program, does not begin until 1942. It should be pointed out, however, that these traffic and revenue

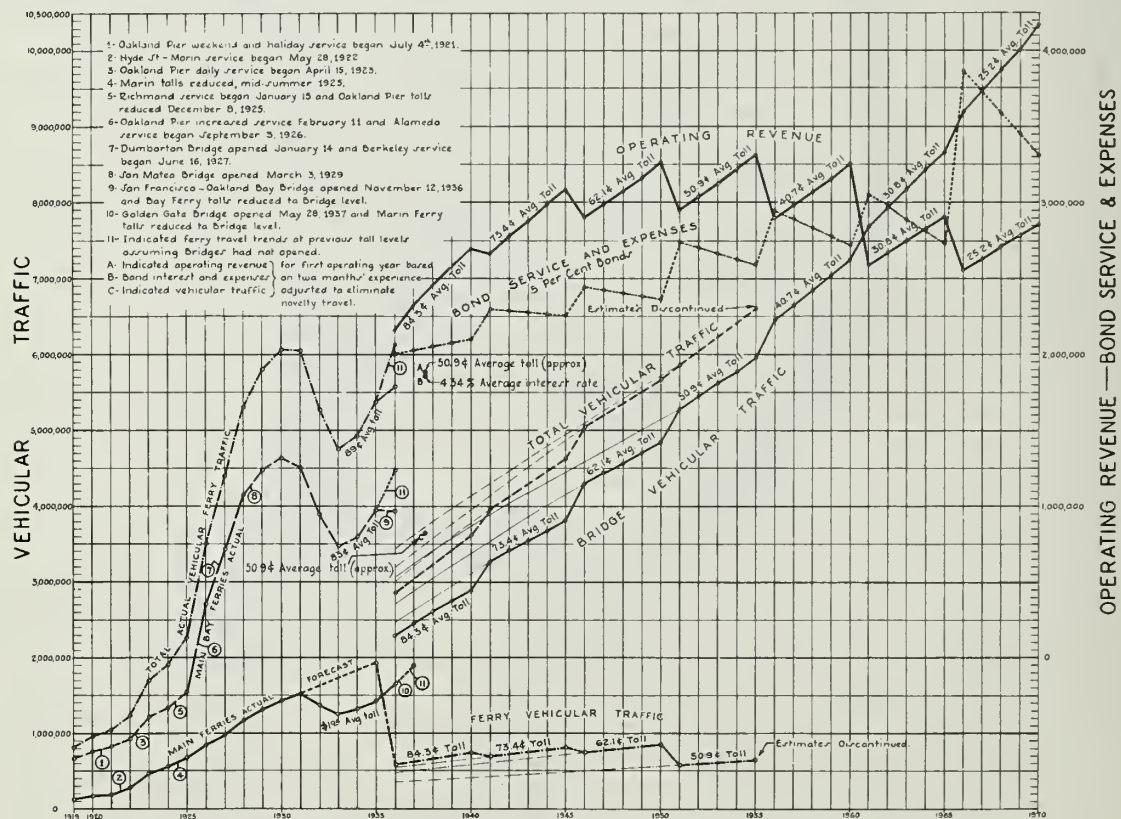


FIG. 2—Forecast of Golden Gate Bridge vehicular traffic and revenue

estimates are based on but two months' experience, and, although the amounts recorded therefor have been liberally adjusted to eliminate the curiosity travel of these first two months, many influences can operate before next August to reduce the bridge's share of the total travel between San Francisco and Marin. As an example, the California Railroad Commission authorized a drastic reduction in automobile rates applicable to the competing lines of the Southern Pacific Golden Gate Ferries, Ltd., and effective the 11th of August, 1937. Since then, both the Golden Gate and the San Francisco-Oakland Bay Bridge have sustained severe traffic losses.

It is of interest to record that truck traffic over all bay crossings (not segregated on the chart) persistently increased throughout the lean years of the depression, notwithstanding the setbacks sustained by the automobile; and that the Marin ferry lines, due in large part to their preeminence as carriers of week-end and holiday travel, and notwithstanding a toll level 25 per cent higher than the prevailing level in effect over the main bay, were the last to succumb to economic pressure and the first to respond to the stimulus of better times. And when the Golden Gate Bridge opened for traffic, the Marin lines having reduced their tolls on automobiles the previous March, were handling traffic at the estimated rate of 2,280,000 vehicles annually, or 47.5 per cent more than their previous all-time high of 1931.

TABLE 2.

GOLDEN GATE BRIDGES AND HIGHWAY DISTRICT

Schedule of Interest and Redemption, July 1, 1937 to July 1, 1971

Year Ending July 1	Series "A" Principal Outstanding	Series "B" Principal Outstanding	Interest at 4-3/4%	Series "C" Principal Outstanding	Interest at 5-3/4%	Total Principal Outstanding	Total Interest Payable	Total Principal Maturing	Total Interest & Redemption	Year Ending July 1
1938	\$ 6 200 000	\$14 500 000	983 250 00	\$14 300 000	\$ 536 250 00	\$35 000 000	\$ 1 519 500	-	\$ 1 519 500	1938
1939	6 200 000	14 500 000	983 250 00	14 300 000	536 250 00	35 000 000	1 519 500	-	1 519 500	1939
1940	6 200 000	14 500 000	983 250 00	14 300 000	536 250 00	35 000 000	1 519 500	-	1 519 500	1940
1941	6 200 000	14 500 000	983 250 00	14 300 300	536 250 00	35 000 000	1 519 500	-	1 519 500	1941
1942	6 200 000	14 500 000	983 250 00	14 300 000	536 250 00	35 000 000	1 519 500	200 000	1 719 500	1942
1943	6 140 000	14 425 000	976 837 50	14 235 000	533 812 50	34 800 000	1 510 650	200 000	1 710 650	1943
1944	6 080 000	14 350 000	970 425 00	14 170 000	531 375 00	34 600 000	1 501 800	200 000	1 701 800	1944
1945	6 020 000	14 275 000	964 012 50	14 105 000	528 937 50	34 400 000	1 492 950	200 000	1 692 950	1945
1946	5 960 000	14 200 000	957 600 00	14 040 000	526 500 00	34 200 000	1 484 100	200 000	1 684 100	1946
1947	5 900 000	14 125 000	951 187 50	13 975 000	524 062 50	34 000 000	1 475 250	400 000	1 875 250	1947
1948	5 790 000	13 980 000	939 075 00	13 830 000	518 625 00	33 600 000	1 457 700	400 000	1 857 700	1948
1949	5 730 000	13 835 000	929 337 50	13 635 000	511 312 50	33 200 000	1 440 650	400 000	1 840 650	1949
1950	5 670 000	13 690 000	919 600 00	13 440 000	504 000 00	32 800 000	1 423 600	400 000	1 823 600	1950
1951	5 610 000	13 545 000	909 862 50	13 245 000	496 687 50	32 400 000	1 406 550	400 000	1 806 550	1951
1952	5 550 000	13 400 000	900 125 00	13 050 000	489 375 00	32 000 000	1 389 500	800 000	2 189 500	1952
1953	5 400 000	13 040 000	875 900 00	12 760 000	478 500 00	31 200 000	1 354 400	800 000	2 154 400	1953
1954	5 250 000	12 680 000	851 675 00	12 470 000	467 625 00	30 400 000	1 319 300	800 000	2 119 300	1954
1955	5 100 000	12 320 000	827 450 00	12 180 000	456 750 00	29 600 000	1 284 200	800 000	2 084 200	1955
1956	4 950 000	11 960 000	803 225 00	11 890 000	445 875 00	28 800 000	1 249 100	800 000	2 049 100	1956
1957	4 800 000	11 600 000	779 000 00	11 600 000	435 000 00	28 000 000	1 214 000	1 200 000	2 414 000	1957
1958	4 590 000	11 080 000	744 325 00	11 130 000	417 375 00	26 800 000	1 161 700	1 200 000	2 361 700	1958
1959	4 380 000	10 560 000	709 650 00	10 660 000	399 750 00	25 600 000	1 109 400	1 200 000	2 309 400	1959
1960	4 170 000	10 040 000	674 975 00	10 190 000	382 125 00	24 400 000	1 057 100	1 200 000	2 257 100	1960
1961	3 960 000	9 520 000	640 300 00	9 720 000	364 500 00	23 200 000	1 004 800	1 200 000	2 204 800	1961
1962	3 750 000	9 000 000	605 625 00	9 250 000	346 875 00	22 000 000	952 500	1 600 000	2 552 500	1962
1963	3 480 000	8 360 000	562 400 00	8 560 000	321 000 00	20 400 000	883 400	1 600 000	2 483 400	1963
1964	3 210 000	7 720 000	519 175 00	7 870 000	295 125 00	18 800 000	814 300	1 600 000	2 414 300	1964
1965	2 940 000	7 080 000	475 950 00	7 180 000	269 250 00	17 200 000	745 200	1 600 000	2 345 200	1965
1966	2 670 000	6 440 000	432 725 00	6 490 000	243 375 00	15 600 000	676 100	1 600 000	2 276 100	1966
1967	2 400 000	5 800 000	389 500 00	5 800 000	217 500 00	14 000 000	607 000	2 800 000	3 407 000	1967
1968	1 920 000	4 640 000	311 600 00	4 640 000	174 000 00	11 200 000	485 600	2 800 000	3 285 600	1968
1969	1 440 000	3 480 000	233 700 00	3 480 000	130 500 00	8 400 000	364 200	2 800 000	3 164 200	1969
1970	960 000	2 320 000	155 800 00	2 320 000	87 000 00	5 600 000	242 800	2 800 000	3 042 800	1970
1971	480 000	1 160 000	77 900 00	1 160 000	43 500 00	2 600 000	121 400	2 800 000	2 921 400	1971
			\$25 005 187 50				\$13 821 562 50			
							\$38 826 750	\$35 000 000	\$73 826 750	

To the student of trends, the persistence of bay-crossing traffic, its instant response to favorable influences, will come as no surprise. San Francisco is a traffic reservoir of considerable potentiality. Prior to the opening of the two bay bridges, the city's north and eastern gateways faced water barriers crossed by out-moded ferries charging excessive tolls. Its southern gateway on the other hand was served by four modern, high-speed highways, available day and night leading down the beautiful peninsula of San Mateo. Result: out of the more than twenty-two million vehicles which were entering or leaving the city each year, only a fifth used the ferries, and of this fifth, less than a third part crossed between San Francisco and Marin.

Bear in mind that motor vehicle registration is just as high in the East Bay Cities as it is in San Francisco, and that the recreational advantages of Marin County and the Redwood Empire are no less varied and attractive than are those of San Mateo and the South. Why, then, was San Mateo travel compared with that through the north and eastern gateways so disproportionately large? The answer lies in the inherent inadequacy and high cost of a ferry service contrasted with the time-saving and freedom of an open road.

And this, too, explains the past response by the automobilist to better ferry service and lower tolls whenever the ferry operators have vouchsafed such concessions to the growing demands of their patronage; for the record over the years since 1929 clearly shows that the ratio of ferry travel to south gateway travel has invariably increased following each improvement in ferry service. This ratio may be taken as a sort of indicator of the potentiality of San Francisco's traffic reservoir. We have seen that the ratio stood at twenty/eighty at the time of the opening of the bridges. Since their opening it has increased approximately to forty/sixty. The bridges drastically reduced all bay-crossing toll rates (See Chart) and cut the travel time enormously between San Francisco and the north and east.

If we measure the circuit distance from Waldo Point southward over the route of the Sausalito-Hyde Street ferry, thence by way of the Marina Approach and back over the bridge to Waldo Point again, the middle point of the circuit will fall at Buchanan Street in San Francisco. Similarly, the middle point of the circuit measured by way of the proposed but uncompleted Funston Avenue Approach instead of the Marina Approach will fall at Steiner Street three blocks west. This narrow strip in San Francisco between Buchanan and Steiner, extending southward between Dolores and Sanchez constitutes the median zone with respect to travel distance between San Francisco and Marin; for from all points within this zone the distance is approximately the same whether we travel by way of the ferry or over the bridge. But the time advantage to the automobilist from points within the zone is 27 minutes in favor of the bridge. Even a horse-and-buggy-owning citizen residing within the limits of this median strip—if there be such—could give his Lincoln-owning neighbor who preferred the ferry a five-minutes' start, and Old Dobbin would beat the Lincoln by two minutes to Sausalito via the bridge.

GENERAL ASPECTS

The Golden Gate Bridge is fortunate in having a scenic setting which is world famous and which provides entry to the structure through the orderliness of a residential section of the city and through the landscaped areas of one of the nation's finest military reservations. The approach to the bridge from either side leaves a pleasing impression upon the visitor, which is heightened by the magnificence of the view from the bridge proper.

The two wide sidewalks and the open type of handrailing with off-set bays at intervals, fulfill the expectation of the designer in providing a promenade of unusual and appealing character. The popularity and inspiring character of these footwalks is evidenced by the tremendous number of pedestrians who daily, and particularly on week-ends, view the Golden Gate from these lofty vantage points.

The Golden Gate Bridge was the first bridge to be built under a district plan. This alone made it possible and brought about the present bridge-minded attitude of the people of San Francisco and the Bay Area. Bearing in mind the agitation for "free bridges" it should be remembered that there are no free bridges, that all bridges must be paid for in taxes of some sort. In the case of the Golden Gate Bridge a toll or users' tax was the only method by which the age-old isolation of San Francisco could be ended.

The toll rate fixed by the Bridge District was modest and fair, and although lower than that upon which the Chief Engineer based his conclusions as to financing, will nevertheless make the bridge a self-liquidating project when all present handicaps are removed. But even were it not a self-liquidating project, the bridge would be worth all its cost and more in the tremendous impetus it has given to the physical unification of the bay area; to the establishment of the beginnings of a genuine metropolitan area; and to the growth and prosperity of San Francisco and its environs. The Golden Gate Bridge, aside from its revenue producing capacity is an asset of incalculable value to San Francisco, California and the nation.

Standing at the portals of the Redwood Empire, and constituting as it does the last link of the great highway from Canada to Mexico, the Golden Gate Bridge, aside from its physical attributes, is California's outstanding contribution to progress and international amity.



PRINCIPAL DIMENSIONS AND QUANTITIES

Total Length of Bridge Including Approach Structure	8,981 ft.
Length of Suspended Structure	6,450 ft.
Length of Main Span	4,200 ft.
Length of Each Side Span	1,125 ft.
Width of Bridge	90 ft.
Width of Roadway Between Curbs	60 ft.
Height of Towers	746 ft.
Clearance Above Mean Lower Low Water	220 ft.
Weight of Main Span Per Lineal Foot	21,300 lbs.
Live Load Capacity Per Lineal Foot	4,000 lbs.
Total Weight on Marin Pier Foundation	264,000,000 lbs.
Total Weight on San Francisco Pier Foundation	726,000,000 lbs.
Weight of Cable Anchorage at Each End of Bridge	240,000,000 lbs.
Deepest Foundation Below Mean Lower Low Water	110 ft.
Maximum Transverse Deflection, Center Span	27.7 ft.
Maximum Downward Deflection, Center Span	10.8 ft.
Maximum Upward Deflection, Center Span	5.8 ft.

TOWERS

Height Above Water	746 ft.
Weight of Two Towers	88,800,000 lbs.
Number of Cells at Base, Per Leg (3'-6" x 3'-6")	103
Number of Cells at Top, Per Leg	21
Base Dimensions (Each Leg)	33 ft. x 54 ft.
Load on Tower from Cables	123,000,000 lbs.
Transverse Deflection	12½ in.
Longitudinal Deflection	} Shoreward—22 in.; Channelward—18 in.

CABLES

Diameter of Cables Over Wrapping	36⅜ in.
Length of One Cable	7,650 ft.
Number of Wires in Each Cable	27,572
Number of Strands in Each Cable	61
Size of Wire (No. 6) Diameter	0.196 in.
Total Length of Wire Used	80,000 miles
Weight of Cables, Suspenders and Accessories	24,500 tons

CONCRETE QUANTITIES

San Francisco Pier and Fender.....	130,000 cu. yds.
Marin Pier.....	23,500 cu. yds.
Anchorage, Pylons and Cable Housings.....	182,000 cu. yds.
Approaches.....	28,500 cu. yds.
Paving.....	25,000 cu. yds.
Total.....	389,000 cu. yds.

STRUCTURAL STEEL QUANTITIES

Main Towers.....	44,400 tons
Suspended Structure.....	24,000 tons
Anchorage.....	4,400 tons
Approaches.....	10,200 tons
Total.....	83,000 tons

LIST OF CONTRACTORS

Main Piers.....	Pacific Bridge Company
Anchorage & Approach Piers.....	Barrett & Hilp
Structural Steel of Suspension Spans.....	Bethlehem Steel Company
Cables of Suspension Spans.....	John A. Roebling's Sons Company
Structural Steel of San Francisco and Marin Approaches.....	J. H. Pomeroy & Co., Inc., and Raymond Concrete Pile Company
Presidio Approach Road.....	Eaton & Smith
Pavement for Suspension Spans and San Francisco and Marin Approaches.....	Pacific Bridge Company and Barrett & Hilp
Electrical Work.....	Alta Electric & Mechanical Company, Inc.
Elevators in Towers.....	Otis Elevator Company
Toll Plaza.....	Barrett & Hilp
Final Paint Coat.....	Pacific Bridge Painting Co.

BOOK TWO



BOOK TWO

PLANNING AND DESIGNING A
BRIDGE WITH 4200-FOOT SPAN,
750-FOOT TOWERS AND A DEEP-
WATER PIER IN OCEAN EXPOSURE

PLANNING

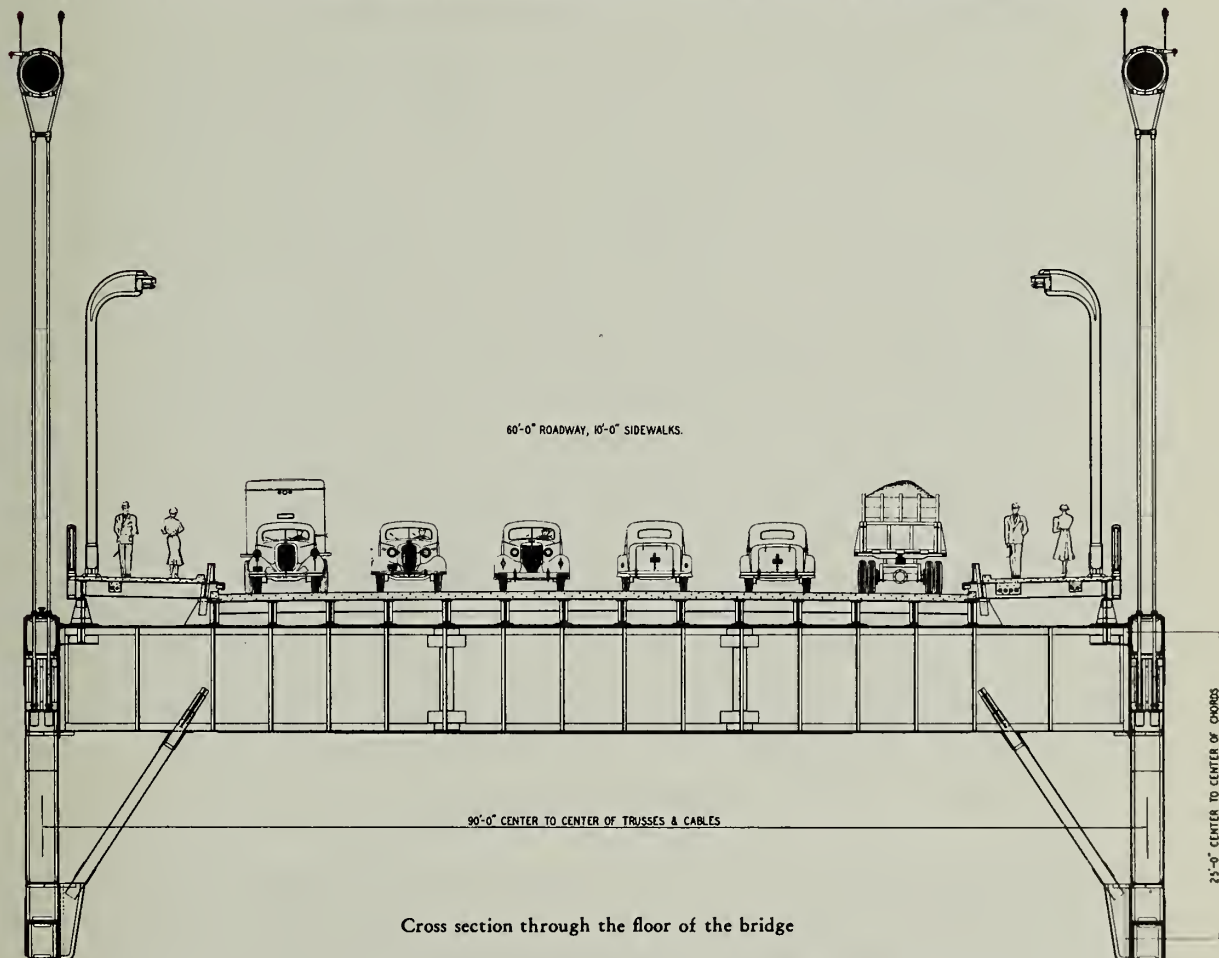
THE development of plans for large engineering projects begins with investigations and studies to determine which one of several possible developments would prove most desirable. Based upon these studies, usually extensive in scope, the major features of the general plan are established and the project assumes definite form. In the case of large bridges the studies normally include consideration of crossings at several different feasible sites from which a choice can be made only after a careful weighing of the merits of each. The procedure in the Golden Gate Bridge project was much simpler because physical conditions at the entrance to San Francisco Bay are such that a bridge joining the two peninsulas should unquestionably be located at the Golden Gate, extending from Fort Point on the South to Lime Point on the North. Obviously, any other location would involve a much longer structure and would introduce additional construction difficulties with no compensating reduction in those present at the site favored.

From the south shoreline, the floor of the strait slopes gently toward the channel for some 1300 feet but beyond that point the water depth increases rapidly. The foundation for the South Pier, desired to be not less than 20 feet into the rock, could be prepared under air pressure at a depth of 100 feet below the surface of the water. Meeting the above prerequisites placed the pier 1100 feet out from shore where the maximum water depth at the channel face of the pier is about 80 feet.

On the north shore, the steep slope of the rock bottom prohibited the placing of piers any appreciable distance off-shore. The length of the main span was thus fixed at 4200 feet since water depth would make the construction cost of piers between these points prohibitive. The natural location of the South Shore-end Pylon at the Fort Point shoreline fixed the length of the side spans at 1125 feet. The narrow point of land which served so well to reduce the length of the structure left little leeway as to the location of the bridge in the east-west direction, but fortunately it was possible to keep the south cable-anchorage on land along the west shore of the Point where a setting in serpentine rock could be prepared for it. On the Marin side the 1125 foot sidespan had to skirt the high cliff and terminate in a Pylon just in front of the north cable-anchorage. The topography at this point is well suited for the gravity type anchorage finally adopted since the natural slope of the rock's surface follows the slope which the cables take and it was only necessary to cut a suitable niche in the side of the hill to receive the mass of anchorage concrete.

The bridge terminates within military reservations on both sides of the Gate and from these terminals, roadways had to be constructed through the reservations joining the bridge with existing streets and highways. The location of these roads so as to minimize interference with military facilities was a major task. On the San Francisco side a route was worked out which anticipated a future branch extending southward

across the Presidio to connect with Funston Avenue. On the Marin side the extremely rugged terrain added to the difficulty of locating a suitable connecting road. The alignment finally selected passed to the westward of Sausalito joining Route 101 at Waldo Point, a distance of $3\frac{1}{2}$ miles. This carries traffic around Sausalito which is desirable. A lateral, branching off to the eastward, connects with Water Street in Sausalito and serves the needs of local traffic. The road to Waldo Point became the obligation of the California State Highway Commission as an extension of State Route 101. The proposed branch road through the Presidio connecting with Funston Avenue became the obliga-



tion of the City of San Francisco and the State who, at the time this report is written, are working out its final alignment with the War Department.

The War Department by its permit of August 11, 1930 established the minimum clear height above mean higher high water, of 210 feet at the piers and 220 feet at the center of the main span. This requirement of the permit, of course, determined the height of the bridge. Low-steel at mid-span under normal temperature of 70° F. and dead load only would have to be at least 236 feet above mean lower low water in order

to give 220 feet clearance above mean higher high water at 110° F. with a live load on the main span averaging 4000 pounds per lineal foot. The sixteen feet difference allows for 10 feet increased sag in the main span due to a 40° F. rise in temperature plus live load and 6 feet for the difference between mean higher high water and mean lower low water. (The latter is used as datum throughout the work). With low steel fixed at elevation 236, the elevation of the roadway and cable at mid-span was readily established at 266.70 and 276 respectively. Choosing a cable sag of 470 feet then fixed the height of towers at 746 feet above datum.

As a matter of proper proportioning it was felt that the width of the bridge having a 4200 foot span should be not less than 90 feet; therefore this was established as the width center to center of cables and stiffening trusses. With tower-shafts centered under the cables there would be room for a 60-foot roadway to pass through the towers. It was evident that full advantage should be taken of the possibility of having at once a six-lane road at a relatively small added cost. This would be more than offset by the relief it would offer during periods of extraordinary traffic which would occur even during the early days of service; however, it seemed desirable to occupy with curbs, sidewalk and balustrade the fifteen foot strip remaining outside the roadway on each side of the bridge. This provided sidewalks having a clear width of 10 feet. Balconies around the outsides of the towers serve there as continuations of the sidewalks.

The lateral system is placed in the plane of the top chords of the stiffening trusses, and at each floorbeam, knee-braces are provided to give lateral support to the bottom chords. Panel lengths are 25 feet and at every second panel point the stiffening trusses are connected with the cables by double suspenders (four parts) of wire rope. The latter are looped over cast steel cable-bands on the cables and their free ends, fitted with forged steel sockets, are attached to the webs of vertical truss members where suitable stiffened seats are provided to receive them. Underneath each sidewalk there is provided a horizontal truss designed to support the curb at six-foot intervals against side thrusts that may be delivered to it by traffic.

BASIS OF DESIGN

Since stresses in the principal parts of the suspension spans depend upon loads applied over great lengths, a proper proportioning of the structure would be obtained only if the basis of design made due allowance for this fact. A single, solid line of vehicles with high percentage of trucks will average in weight about 500 pounds per lineal foot. Such a load in each of the six traffic lanes, making a roadway load of 3000 pounds per foot of bridge, obviously is a congested loading not apt to occur, but one which may be approached should traffic for any reason be blocked from leaving the bridge. In such event cars will not stand in contact end to end but will be separated by short open spaces which will, of course, materially reduce the load per foot. The two ten foot sidewalks may receive loads of 100 pounds per square foot over small areas but an average of 50 pounds

per square foot is ample when applied throughout the structure. Accordingly it was recognized that 1000 pounds per lineal foot would suffice for the sidewalks. The basic live load for the suspended spans was therefore taken at 4000 pounds per lineal foot of bridge. All loads used for design of the Main Structure are as follows:

LIVE LOAD: The basic live load for towers, cables and stiffening trusses is 4000 pounds per lineal foot of bridge and of the length and position to produce maximum stress. Live load stresses for combination with wind stresses from a wind load of 30 pounds per square foot were computed for the above live loading except that in the case of the center span stiffening trusses the live load was taken at 2000 pounds per lineal foot uniformly distributed over the entire center span plus 2000 pounds per lineal foot of any length and position in the span to produce maximum stress.

The roadway slab and stringers were designed to carry 24 ton trucks having wheels spaced six feet gauge and axles nine feet center to center. Two thirds of the load was assumed to be on the rear axle. Impact was assumed to equal 50% of the live load. The sidewalk slab and stringers were designed for a live load of 100 pounds per square foot.

Floorbeams were designed for roadway load of six 24-ton trucks abreast with a reduction factor of 75% to allow for the large number of loaded lanes. Impact was assumed to equal 25% of the live load. In addition to the roadway load a uniform live load on the sidewalks amounting to 100 pounds per square foot was included.

WIND LOAD: Wind pressures used in design were 30 pounds per square foot on the cables and suspended structure and 50 pounds per square foot on the towers. The exposed area used for computing transverse wind pressure was taken as twice the vertical projection of the structure as seen in side elevation. The exposed area used for computing longitudinal wind pressure for the suspended structure was taken as one half the vertical projection of each web member of the stiffening trusses and of each floor beam and knee brace.

TEMPERATURE: The range of temperature has been assumed to extend from 30° to 110° F. Normal temperature has been taken at 70° F.

DEAD LOAD: The dead load of the center span if uniformly distributed would be as follows:

	<i>Pounds per lineal foot</i>
Cables, suspenders and accessories.....	6,670
Stiffening trusses	3,330
Floor system, curbs and railings.....	3,830
Bracing	600
Concrete paving	6,470
Conduits and miscellaneous.....	400
Total as built.....	21,300

The dead load in the sidespans as built is 21,500 pounds per lineal foot. For purpose of design an assumed uniform dead load of 21,000 pounds per lineal foot was used for all spans.

UNIT STRESSES: Permissible unit stresses used in the design are as follows:

TOWERS

	<i>Lbs. per sq. in.</i>
Direct stress in shafts from dead load, live load and temperature changes	
Carbon steel	14,000
Silicon steel	18,000
Combined direct and bending stresses in shafts—	
Carbon steel	18,000
Silicon steel $\frac{7}{8}$ in. plates	20,000
15/16 in. plates	23,000
Maximum	24,000
Bearing, carbon steel	24,000
Bearing, silicon steel	30,000
Bearing, power driven rivets	24,000
Bearing, carbon steel pins, heat treated	27,000
Shear, carbon steel	10,000
Shear, silicon steel	13,000
Shear, power driven rivets	12,000
Shear, carbon steel pins, heat treated	13,500
Tension, power driven rivets	5,000

SUSPENDED STRUCTURE

Stiffening truss chords, silicon steel	
Tension	32,000
Compression	25,000
Stiffening truss web members, carbon steel	
Tension	21,000
Compression	21,000— $90 \frac{1}{r}$

Where l = length in inches of the unsupported length of the member, and r = the radius of gyration in inches.

Lateral Bracing, silicon steel	
Tension, including participation	32,000
Compression, including participation	25,000
Rivets for stiffening trusses and bracing	
Bearing, power driven	27,000
Shear, power driven	15,000
Floorbeams and Stringers	
Tension, carbon steel	16,000
Tension, silicon steel	22,000
Compression, carbon steel	16,000— $150 \frac{1}{b}$
Compression, silicon steel	22,000— $200 \frac{1}{b}$

Where l = the length in inches of the unsupported flange, and b = the flange width in inches.

Shear, carbon steel	10,000
Shear, silicon steel	13,000
Bearing, power driven rivets	24,000
Shear, power driven rivets	12,000

All Other Structural Members	<i>Lbs. per sq. in.</i>
Tension, carbon steel.....	16,000
Tension, silicon steel.....	24,000
Tension, heat treated eyebars.....	27,000
Compression, carbon steel.....	16,000—70 $\frac{1}{2}$ (14,000 Max.)
Compression, silicon steel.....	24,000—100 $\frac{1}{2}$ (18,000 Max.)
Bearing, carbon steel.....	24,000
Bearing, silicon steel.....	30,000
Bearing, power driven rivets.....	24,000
Bearing, carbon steel pins, heat treated.....	27,000
Shear, carbon steel.....	10,000
Shear, silicon steel.....	13,000
Shear, power driven rivets.....	12,000
Shear, carbon steel pins.....	13,500
Tension, power driven rivets.....	5,000
Bending on extreme fibres carbon steel pins, heat treated.....	27,000
Steel Castings	
Bending on extreme fibres.....	15,000
Bearing.....	24,000
Direct compression.....	20,000
Cable Wire, tension.....	82,000
Heat Treated Bolts, tension.....	22,000
Bronze, bearing.....	3,000

The materials used were required to have yield points and ultimate strengths not less than the following, in pounds per square inch:

	<i>Minimum Yield Point</i>	<i>Minimum Ultimate Strength</i>
Rolled carbon steel.....	36,000	60,000
Rolled silicon steel.....	45,000	80,000
Rivet Steel.....	30,000	52,000
Heat-treated pins and bolts.....	60,000	95,000
Forged steel rollers.....	40,000	80,000
Cast steel.....	35,000	65,000
Heat-treated eyebars.....	50,000	80,000
Cable wire.....	160,000	220,000

The Preliminary Plans which formed a part of the Chief Engineer's Report of August 27, 1930 were tentative in character and were developed only to the extent required to insure the sufficiency of the cost estimate and to present one feasible layout which would meet the requirements of the War Department as well as the needs of the District. The major dimensions of the main structure were necessarily determined, as were also, definite locations for the main piers. Many features of the design shown were



of course subject to whatever modifications a thorough and comprehensive engineering study would develop. For example, tunnel type anchorages for the cables were shown on the Preliminary Plans but subsequent study led to the adoption of gravity type anchorages.

MAIN PIERS

Preliminary foundation studies completed in 1930 and covered by the Chief Engineer's Report of August, 1930, were based upon core borings taken on each shore and at the proposed pier sites. These borings, eighteen in all, showed that the North Pier at Lime Point would be founded on basalt containing inclusions, large and small, of radiolarian chert, an adjoining formation into which it was intruded. The rock is very strong and the cores showed it to be uniform in character except for the occasional inclusions of chert which do not in any way detract from its strength.

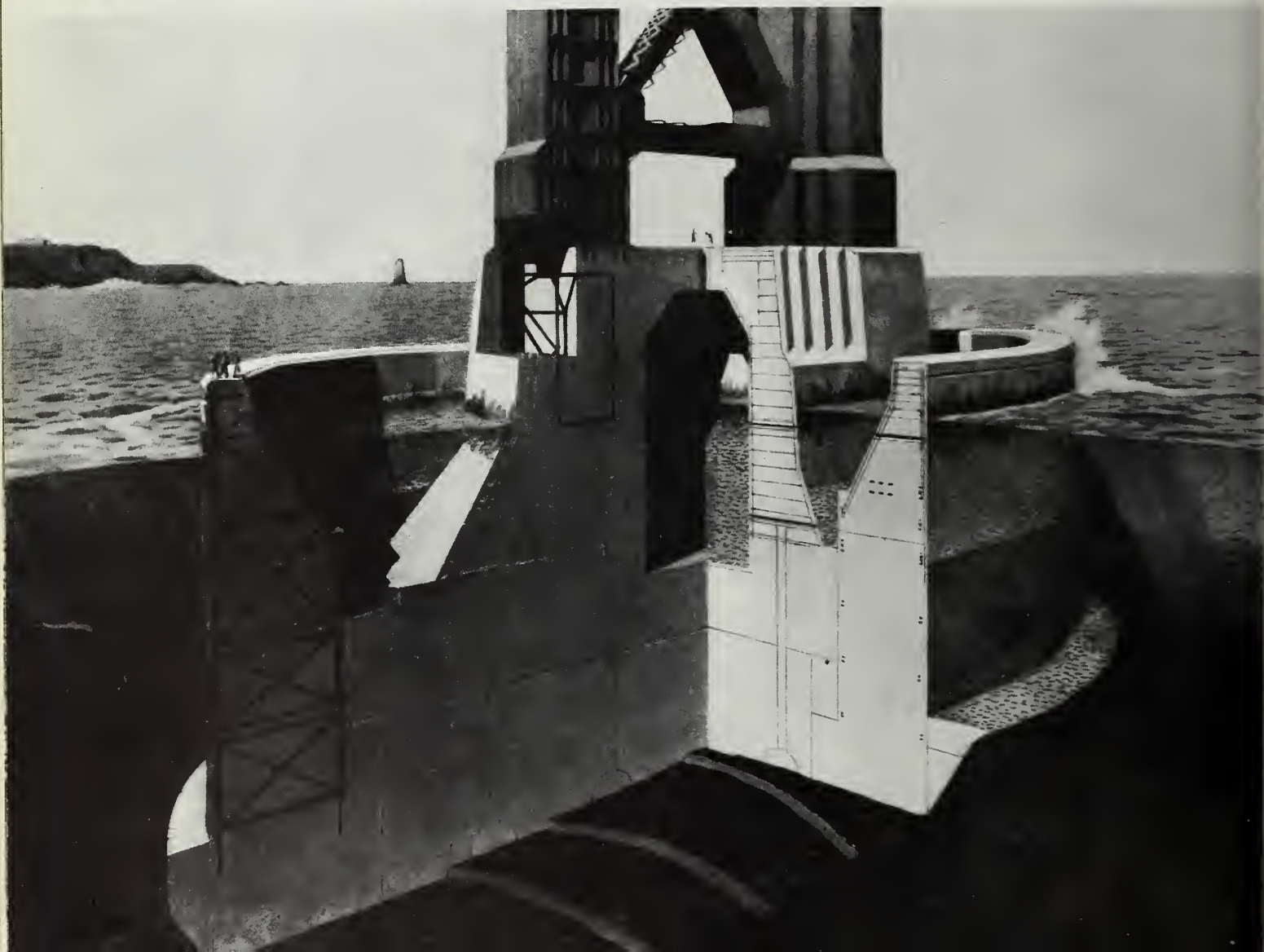
Borings at the site of the South Pier showed the rock to be of serpentine identical with that exposed on the shore at Fort Point and having a bearing strength not less than that of the rock upon which the previously described bearing test was made. Subsequent to the 1930 borings fourteen additional core borings were taken at this pier site. In most instances the borings penetrated the rock for 60 or 100 feet and in one case for 160 feet to elevation -251.6. The borings showed conclusively that the serpentine rock foundation for the South Pier was uniform in character for a minimum depth of 160 feet which corresponds to an elevation of 250 feet below sea level.

The design of the Marin Pier, located on the shoreline as it was, involved no noteworthy engineering problem. A U-shaped cofferdam built out from shore about the pier site would permit the excavation of the rock and preparation of the pier foundation at elevation -20 and the subsequent construction of the pier, to be carried on as a land operation. The pier base was made 80 feet wide and 160 feet long, the length being determined as the minimum which would accommodate the base dimensions of the tower and the width as the minimum required for stability. The maximum pressure on the base with full live load on all spans is 132,000 tons of which 48,000 tons is due to the weight of the pier itself. The resulting average unit pressure is 10.7 tons per square foot. With full live load on the center span and the far side span, at highest temperature, and with transverse wind, the maximum unit pressure on the base of the pier is 17.1 tons per square foot and the minimum is 4.0 tons per square foot.

The development of plans for the San Francisco Pier involved consideration of extraordinary conditions at the site. Attention has already been called to the severe exposure to ocean storms and tidal currents and bare-rock bottom. The pier must be built at a site exposed to the full force of ocean storms where the bare-rock bottom in 65 feet of water is swept by tidal currents of 6.5 knots per hour. Early in the study of the problem the Chief Engineer concluded that the permanent protective fender which would be required around the pier in the completed structure should be designed so

that it could be built prior to the pier. Construction of the pier might then proceed in quiet water within the fender. This novel procedure had been developed sufficiently during the preparation of the Preliminary Plans so that those plans showed a complete fender ring comprising two steel plate walls 10 feet apart carried on structural steel frame work somewhat like a floating drydock. The plan contemplated that this would be built at a shipyard, launched and floated to the site where it would be lowered on to the rock bottom and the 10-foot space filled with concrete. As the problem was further studied in the preparation of the Bidding Plans it became more and more evident that the general scheme of building the fender first and using it to facilitate the building of the pier, was sound. There were, however, many objections to the plan as then developed. The big steel, hull-like structure would be costly to build; difficult to handle; uncertain as to shape of bottom to fit contours of the rock surface upon which it would be sunk; the necessary bracing spanning the space enclosed, would interfere with the sinking of a pneumatic caisson; the 10 foot wall thickness would probably need to be increased. These very objections pointed out the way to the solution. The fender construction should not begin with the placing of a large fabricated unit but should be built up in place starting with a small initial unit to which other units would be added successively.

Cut-away section
showing construction
of San Francisco
Pier and Tower



In order to minimize the interference of tide and waves the initial unit would start with a skeleton steel frame about 30 feet square in plan, with columns at each of the four corners. This frame would be securely anchored into previously drilled anchorage holes. Sheet piling or forms would then be placed around the lower section and it would be filled with concrete. With the added stability thus obtained the next section above could be formed and it filled with concrete. After the initial unit was completed adjacent units could be built in much the same manner. Thus as the building of the fender progressed its stability would be ever increasing at a rate equal to or greater than the rate of increase of area exposed to current and wave. It was conceived that work could thus go forward in both directions from the initial unit until the enclosure was completed. When completed there would be a concrete wall 30 feet thick all around the pier site. A pneumatic caisson could then be constructed within the walls of the fender and sunk to the desired depth into the rock.

In accordance with this conception the San Francisco Pier and Fender design was developed and shown on the Bidding Plans. In order to give the greatest possible latitude to the contractor the plans for the fender showed its design as a completed unit in all essentials with no limitations as to methods of construction except that under-water concrete must be placed by the tremie method and in sections not over 40 feet in length. For the pier these plans showed a base 90 feet wide and 185 feet long with its bottom surface at elevation -100. As construction progressed modifications were made in the plans to meet difficult situations as they arose. These are described in Book Three but it will be noted here that as finally built the fender and pier proper are combined as a single structure below elevation -35. The total area of the base thus formed is 37,800 square feet. Pressure on this base is 363,000 tons, of which 279,000 tons is in the weight of the pier and fender. Live load on the bridge contributes only 1.9 per cent of the total pressure. The average unit pressure is 9.6 tons per square foot and under the most severe combination of load, temperature and wind the maximum unit pressure is 10.8 tons per square foot.

An earthquake acceleration equal to 10 per cent of the acceleration of gravity, which is greater than can reasonably be anticipated, would increase the maximum bearing pressure on the Marin Pier by not more than four tons per square foot and that on the San Francisco Pier by not more than three tons per square foot.

ANCHORAGES

Although the Preliminary Plans showed tunnel type anchorages on both sides of the Gate, subsequent study showed that it would be unwise to subject the rock on either side to the character of stress which would result from the use of that type of anchorage. Accordingly gravity anchorages were developed and shown on the Bidding Plans. These anchorages comprise, at each cable-end, a mass of concrete, weighing approximately 60,000 tons set well down into the rock foundation. They are about 150 feet

long and have embedded within them heavy eye-bar chains which terminate in a series of upright girders at the rear ends of the anchorages. At the forward end of each anchorage the eye-bar chains protrude from the concrete as 61 pairs of bars arranged to receive the 61 strands which make up the cable. At each cable-end the total cable pull of 62,000,000 pounds is delivered to the anchorage through these 61 strands. On the San Francisco side, at the anchorage, the cable curve has an inclination of $16^{\circ} 31'$ with the horizontal. On the Marin side the corresponding inclination is $17^{\circ} 40'$. These angles therefore give the direction of application of the cable-pull. The 60,000 ton weight of the anchorage mass is sufficient to turn this pull into a resultant thrust downward against the rock foundation with an inclination from the vertical of about 30° . The unit bearing pressure over any portion of any anchorage foundation does not exceed 10 tons per square foot. The disposition of the anchorage mass is such that it would be stable even though the cable pull were increased to twice the design load.

The anchorages are enclosed in reinforced concrete housings, one for the San Francisco side and one for the Marin side. The roof of the housing on the Marin side serves also as the floor of the bridge.

The eye-bar chains which transmit the loads from the cables to the anchorages are arranged in three tiers of 122 bars each. Those of the first tier are pin-connected to the nine upright girders at the rear of the anchorage. Each girder except the center one receives 14 eye-bars, in three groups, 4 at each end and 6 at the center. The eye-bars are heat treated. Those of the first tier are 10 in. by 2 in. and 33 ft. 4 in. long. Those of the second tier are 10 in. by 2 in. and 66 ft. 7 in. long. Those of the third tier (except for two pairs) are 10 in. by $2\frac{1}{8}$ in. and 33 ft. 4 in. long.

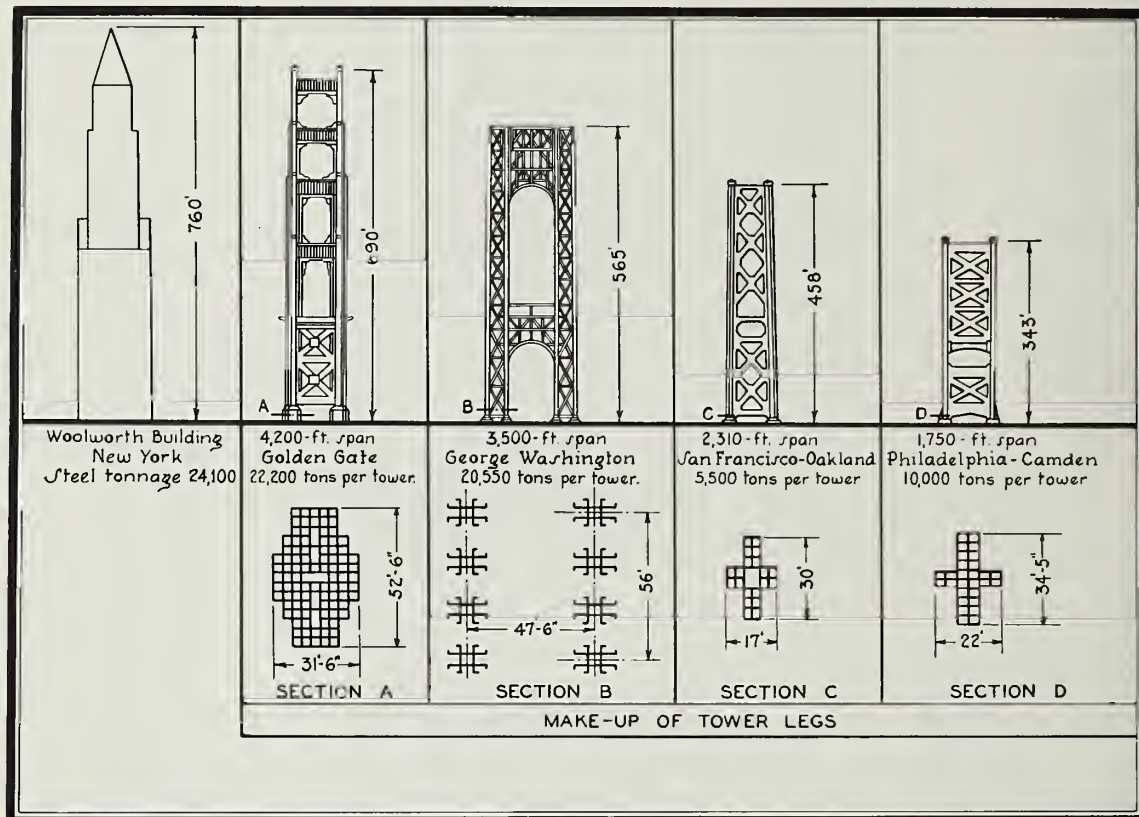
Since the aim was to deliver the cable-pull to the girders at the rear end of the anchorage, precautions were taken to minimize the tendency of the embedded eye-bars to unload through bond with concrete as well as by direct bearing of eye-bar heads against the concrete at connections of adjoining bars. Edges of such eye-bar heads were wrapped with mastic $1\frac{1}{2}$ -in. in thickness. While it was felt that painting the eye-bars with some covering which would prevent bond with concrete was highly desirable, a suitable material, known to be free of possible injurious effect over a long period of time, was not available. The bars were given two coats of heavy red lead paint.



MAIN TOWERS

*From a paper by Clifford E. Paine published in
Engineering News-Record October 8, 1936.*

The steel towers supporting the cables, obviously notable because of their size, also command attention because of the unusually close correlation of architectural and structural requirements that characterized their design. Developed as rigid frames made up of two shafts and cross bracing without diagonals in the panels that comprise the upper 500 ft. of their 700-ft. height, the towers each contain 22,200 tons of steel, about



Comparison of Golden Gate Bridge towers with the towers of other notable suspension bridges

10 per cent more than was used in the George Washington Bridge towers at New York. The main section at the base of each shaft or leg of the towers has an area of 12,391 sq. in. as compared with 4500 sq. in. in the Philadelphia-Camden bridge towers of similar make-up and of 5000 sq. in. in the eight columns that comprise one leg of the George Washington Bridge towers. The Golden Gate tower shafts are thus by far the largest structural steel members ever assembled.

The towers are 690 ft. 4 in. high from the piers to the base of the saddle castings, and rise 746 ft. above the water to the intersection of the main and the side span cable tangents. The center lines of the tower shafts are 90 ft. apart, the distance selected as

the desired spacing between centers of cables. This spacing and the necessary width of the tower shafts is just sufficient to provide clearance for the 60 ft. roadway through the towers at deck level.

Supporting a 4200-ft. center span and two 1125-ft. side spans through two $36\frac{3}{8}$ -in. diameter cables, the towers are designed to resist forces of great magnitude caused by dead and live loads, wind and earthquakes. This fact, coupled with the special indeterminate nature of the structures made the design problem a difficult one. Among the contributions to structural practice that came out of the design is a novel application of the Williot diagram, by which strain equations can be written directly without first knowing the stresses that cause the strains.

The general features of tower design were developed concurrently with the architectural treatment, to insure a satisfactory architectural effect without undue cost or structural sacrifice. The architectural treatment is in general modernistic and involves the exclusion of applied decoration and unnecessary elements. Where ornament was desirable, either to enrich certain surfaces or to diversify surfaces unlikely to be perfectly executed over large areas, a simple vertical fluting was used consisting of intersecting plane facets which could be carried out readily in both concrete and steel. Since an ornamental elaboration of minor details such as hand rails, lighting units, etc., is likely to be inharmonious in a structure predominantly engineering and industrial in character, these features were handled in the spirit of the large design, using principally structural steel shapes.

Where great heights are involved, the common architectural practice of enlarging the scale of details as their distance from the ground increases, tends to emphasize the detail as such at the expense of the whole mass. The opposite principle was employed on towers of the Golden Gate Bridge; the scale of details on the towers decreases with the height.

The main shafts of the towers are built up of plates and angles arranged in cellular form (Fig. 3), using sections $3\frac{1}{2}$ ft. square. At the base in the longitudinal direction of the bridge, each shaft is 15 cells wide and in the transverse direction, 9 cells wide. The base slab on which each shaft stands is a 33 x 54-ft. steel plate 5 in. thick, made up of 19 separate pieces. As the shafts progress from the base toward

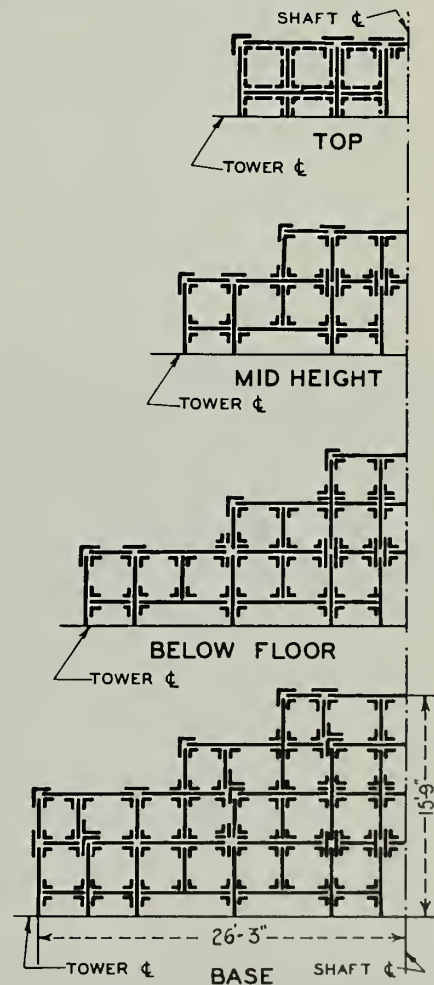


FIG. 3—Cellular makeup of tower shafts sets a new record for steel assemblages; at the base each tower shaft contains 12,391 sq. in.

the top, the section is decreased by successive omissions of outside cells, until at the top the total number of cells is only 21, arranged in the form of a rectangle composed of 3 cells in the transverse direction and 7 cells in the longitudinal direction.

Below the bridge floor level, transverse bracing between the two shafts is provided in the form of two panels of diagonals. Above the floor level, however, transverse bracing is confined to four portal struts. All bracing members have four webs which are spaced to permit gusset connections direct to web plates of tower shafts; thus the width of bracing members is approximately the width of three cells, or $10\frac{1}{2}$ -ft. The portal struts are cased in steel plate enclosures, detailed to carry out the architectural treatment.

FACTORS THAT INFLUENCED DESIGN: The principal architectural influence on the structural make-up of the towers is evident in the portal strut bracing. This form of bracing was found to be efficient, and did not involve any structural difficulties; the spacing of the portal struts was arbitrarily chosen to give the best appearance.

The dead load of the towers is 22,260,000 lb. per shaft. The dead load concentration from the cables on the saddles amounts to 52,000,000 lb. and the live load and temperature concentration to 9,438,000 lb. per shaft. The towers, cables and stiffening trusses were designed for a live load of 4000 lb. per lineal foot of any length and position.

Transverse wind forces applied to the tower were computed on the basis of 30 lb. per sq. ft. on twice the vertical projection of trusses and cables plus 50 lb. per sq. ft. on twice the vertical projection of the tower. For longitudinal wind forces computations were on the basis of 50 lb. per sq. ft. on the vertical projection of the tower plus 30 lb.

Footwalk Ropes
in place



per sq. ft. on one-half the vertical projection of each web member of stiffening trusses and of each floorbeam and knee brace. Temperature stresses were figured for an assumed rise or fall in temperature of 40° F.

At the outset it was decided that the tower shafts were to be of cellular construction formed of 42 x $\frac{7}{8}$ -in. web plates and 8 x 8 x $\frac{5}{8}$ -in. angles. Silicon steel is used from the base up to the roadway and in the top two panels; the two center panels utilize carbon steel. Whenever unit stresses in silicon steel were found to be in excess of 20,000 lb. per sq. in., the thickness of the plates was increased to $\frac{15}{16}$ in. This occurs, for the most part, in the outer cells where bending stresses are of more consequence. Also it was decided at the outset that the l/r ratio of the tower shafts should not exceed 60. The establishment of these proportions pretty well determined the number and arrangement of cells.

The carbon steel specifications required tensile strength of 60,000 to 70,000 lb. per sq. in. and a minimum yield point of 36,000 lb. per sq. in.; for the silicon steel a tensile strength of 80,000 to 95,000 lb. per sq. in. and a minimum yield point of 45,000 lb. per sq. in. were specified.

The allowable unit stresses adopted were as follows:

Direct stress in shafts from dead and live loads and temperature changes:

Carbon steel.....14,000 lb. per sq. in.

Silicon steel.....18,000 lb. per sq. in.

Combined direct and bending stresses in shafts:

Carbon steel.....18,000 lb. per sq. in.

Silicon steel..... $\frac{7}{8}$ -in. plates—20,000 lb. per sq. in.

" " $\frac{15}{16}$ -in. plates—23,000 lb. per sq. in.

" " maximum—24,000 lb. per sq. ft.

LONGITUDINAL STRESSES: The towers are deflected 6 in. toward the shore in their dead load, normal temperature, position. From changes in the sags of the side span cables and from changes in length of the side span cables due to temperature and loading, the tower tops may move to positions 18 in. channelward or 22 in. shoreward from vertical lines through the centers of the tower bases. These movements, determined by the most unfavorable positions of the full live load of 4000 lb. per lineal foot of bridge and a maximum temperature change of 40° F. up or down from normal, are smaller than on some shorter spans because of the favorable ratio of side span to main span, the low proportion of live load to dead load, and the moderate temperature change. Despite the limitation of the longitudinal radius of gyration to $\frac{1}{60}$ of the height of the tower, the maximum unit bending stress from a deflection of 22 in. is only 6000 lb. per sq. in. Under such deflection the bending moment in the shaft at the base is slightly over 3 billion in.-lb.

The analysis of stresses is simple and conventional but the important effect of the eccentricity of the direct loads in reducing, by more than one-half, the amount of

horizontal force from the cables required to produce the desired deflection necessitated several sets of computations before the assumed deflection curve agreed with the computed curve.

Several cases of longitudinal wind combined with longitudinal tower deflection were also studied. However, except for a short distance near the middle of the tower, the longitudinal wind forces did not affect the design of the tower.

TRANSVERSE WIND STRESSES: The transverse wind load per shaft amounts to 3,149,000 lb., of which 30 per cent is from the cable, 26 per cent from the laterals of the suspended spans, and 44 per cent is the wind on the tower itself. Since the history of the design of the towers is practically the history of the determination of the transverse wind stresses, some description of the various methods used to analyze the wind stresses is pertinent.

Determination of the superimposed loads, the longitudinal tower deflections, the maximum slenderness ratio, the unit working stresses and an outline of the structure completed the preparatory steps. The direct stresses from dead load and live load were then tabulated and a guess made at the amount of unit bending stresses from longitudinal movements. The wind stresses were computed on the basis of a point of contraflexure at the center of each unbraced panel. This information was sufficient to permit an approximate determination of the various sections of the shafts, struts and other web members.

Using the tentative sections selected, the longitudinal bending moments were computed and the sections revised to correct the errors made in guessing the amount of the unit stresses from longitudinal bending.

To facilitate investigation of the tentative design under transverse forces, the tower was cut at the floor, and the upper part of the tower figured as an independent structure, consisting of a multiple-story portal frame. The solution assumed identical deflections and changes in slope for the two tower shafts. The fully-braced lower section of the tower was considered as a simple truss. This analysis gave somewhat different results than the first determination of wind stresses, and the sections were again revised.

The methods used so far were very crude in their determination of the stresses. For a more accurate analysis conventional slope deflection methods were next employed. It was assumed that the direct wind stresses previously found were correct; that the struts had no depth (Fig. 4); and that the base of the tower was free to rotate. A conventional Williot diagram was drawn and the usual slope deflection equations were set up and solved to determine the moments in the towers. Direct wind stresses resulting from this solution were substituted for those assumed and a new solution carried out. After several attempts the calculated direct stresses agreed fairly well with those assumed. Sections were revised, longitudinal moments recalculated, sections again revised and the transverse wind stresses again computed. This completed what may well be termed the "Preliminary Design" which was shown on the Bidding Plans.

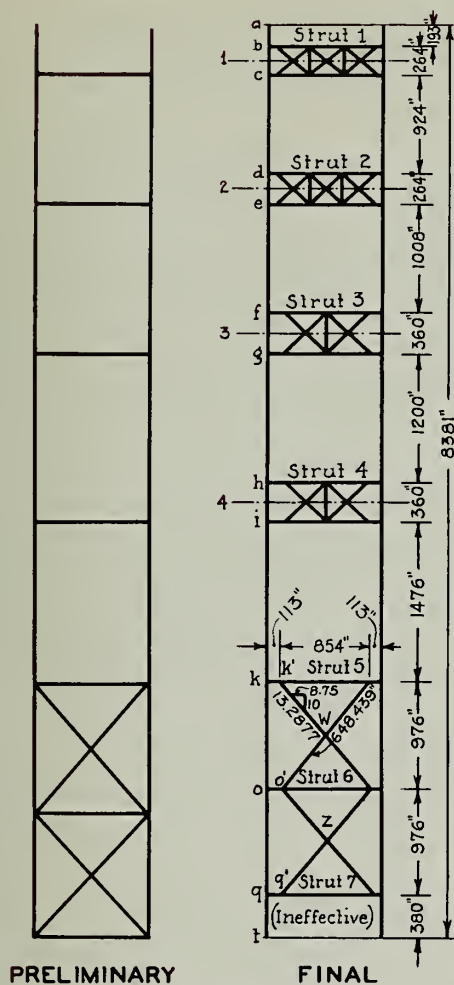


FIG. 4—Tower diagrams used for preliminary and final calculations.

REFINEMENT OF PRELIMINARY DESIGN:

At this stage a critical study of the tower design was made. Some objectionable features were embodied in the preliminary design. Webs, for example, in the longitudinal and transverse faces of the shafts varied in thickness from $\frac{7}{8}$ to 3 in. The splices for webs of 3 in. thickness were long and the riveting burdensome. The gussets for the web members below the floor were very long and the bending stresses in the gussets became large when the intersections of the working lines of the horizontal and diagonal web members were placed on the center of the shaft. Some of the portal struts had short heavy members and the secondary stresses in these members certainly would be high.

The basic assumptions used thus far in the stress determination were far from satisfactory. Obviously an assumption of fixity for the bases of the shafts was more nearly correct than the previous assumption of a free base. The vertical dimension of the struts would have a considerable effect and must be considered. Also the shear carried by the shafts would have to be considered, to make any accurate determination of the direct stresses in the diagonals below the floor.

A survey of the preliminary design indicated that if proper consideration were given to the depth of the struts the main web plates in the shafts could be kept at $\frac{7}{8}$ -in. thickness, although

in certain places they would have to be made of silicon steel, and in a few instances reinforcing plates between the angles would have to be added. This would produce a much cleaner design. There was no reason why the intersection of the working lines of the web members needed to be on the centerlines of the shafts if account were taken of the resulting stresses from the eccentricity. Also, the strut sections and their panel lengths were revised in an attempt to keep the anticipated combination of primary and secondary unit stresses within permissible limits.

The errors of assumptions and analysis used in the determination of the stresses in the preliminary design were next to be eliminated. A correction of the previous calculations showed that the assumption of fixed bases gave moments of considerable magnitude in the shafts below the floor. The shears accompanying these moments were of such magnitude as to affect considerably the stresses in the web members. Also the

appreciable moments resulting from eccentric connections of the web members below the floor had to be included in the calculations. Thus the part of the structure below the floor changed from a very simple truss to a statically indeterminate structure. Study of this perplexing problem led to a clear-cut analysis.

Unique Use of Williot Diagram: In all previous work with slope deflections the primary stresses had been obtained first without regard to the bending stresses, and then a Williot diagram was drawn using the strains derived from the primary stresses to obtain the deflection angles of these web members. It was now realized that the Williot diagram was the missing key to a satisfactory, direct solution. A Williot diagram was sketched without knowing the stresses but from which it would be possible to evaluate the deflections in terms of the various unknown stresses. The unknown stresses could be readily expressed algebraically in terms of the known loads and the unknowns of the slope deflection formulas. This procedure took care of the major difficulties. Placing the angle of rotation of the shaft at the base equal to zero provided the necessary fixity of the base of the shaft, and the inclusion of the eccentric moment from the diagonals below the floor in the summation of moments at the various points took care of those stray moments. Thus the lower part of the structure was adequately analyzed although the solution had to await the necessary equations for the portal-braced, upper portion of the tower.

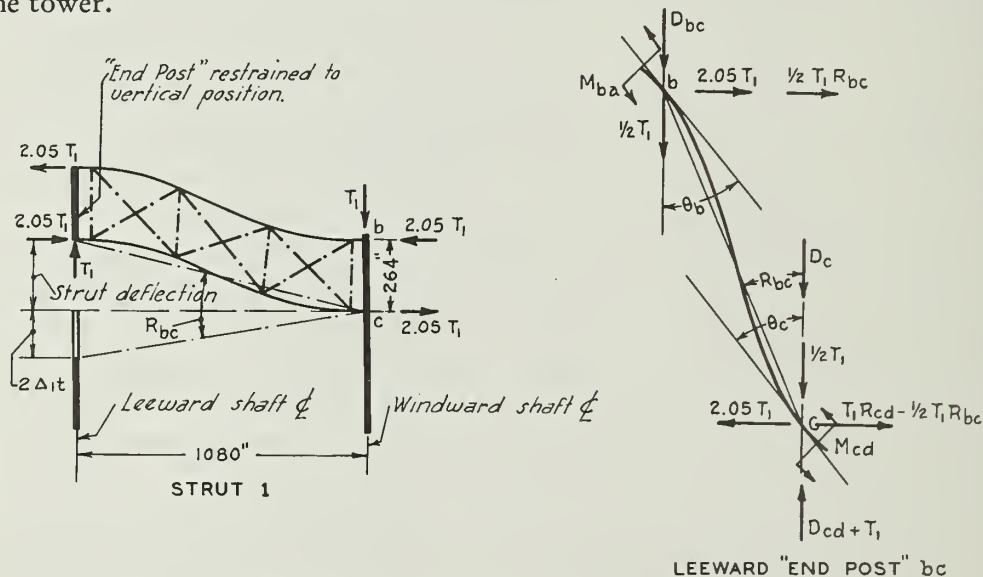


FIG. 5—Shaft Deflection Angle between chords of struts was expressed in terms of the strut deflection from a vertical upward load equal to the shear T_1 and the relative change in length of the two shafts, $2\Delta_{gr}$. The sketches portray the movements of the members involved.

Taking Account of Strut Depth: Transferring attention now to the upper portion of the tower, a possibility of treating the struts in a manner similar to that developed for the part below the floor was considered. However, it was soon evident that the number of equations would become unwieldy. Also it was noted that the secondary moments in

the strut members were far too small to affect the moments in the shafts more than a fraction of one per cent. Apparently the effect of secondary stresses in the struts could be handled adequately without combining this study with the stresses in the structure as a whole.

This decision left the introduction of strut-depth into the analysis as the major remaining problem in this part of the structure. It was found that by using a special equation for the deflection angle of the shaft between the chords of a strut, a satisfactory analysis could be made. This method, described below, is believed to be unique.

First, assume that the end posts of a strut—the shafts between the chords of the strut—are restrained so that their axes remain vertical while the leeward end of the strut is permitted to deflect upward under the action of an upward force equal to the shear in the strut (Fig. 5). If now the strut, including its end posts, is rotated until the leeward end is again at its initial elevation, the end posts will have rotated through an angle equal to the deflection of the strut divided by the distance between the posts. The deflection angle, due to the distortions within the strut as above determined, is increased by the change due to the shortening of the leeward post and the lengthening of the windward post. This added deflection angle is equal to the difference in elevation of the two ends of the struts divided by the distance between the posts.

The shear in the struts was expressed in terms of the known loads and the unknown deflection angles and rotation angles of the shafts. To eliminate the necessity for repeated solutions the elongations and shortenings of the shafts from direct wind loads were also expressed in terms of the known loads and unknown angles.

This method contains an approximation in the assumption that the end post of the strut does not change length. It was assumed, in computing that part of the deflection angle of the shaft between strut chords due to changes in length of the shafts, that the shaft changes in length were from the base of the shaft to the center of the strut. This assumption produced only a small error in the structure as a whole. Account was taken of the changes in length of the shaft between chords when studying the secondary stresses in the struts.

The analysis of the towers was finally concluded with a set of 33 simultaneous equations. Since most of these equations did not contain more than half a dozen unknowns, the solution was not difficult. Having the necessary distortion angles it was possible to compute in the customary manner all of the required stresses from transverse wind.

The moments in the shafts were, of course, appreciably smaller than in the previous calculations. This resulted in the anticipated savings in material, and, together with a few changes to silicon steel permitted the web plates to be of uniform thickness except in a few places where the stresses were sufficiently high to require 15/16-in. plates as a precaution against web buckling. At the base of the shafts the plates were thickened to 15/16-in. to resist corrosion. The transverse deflection of the tower by this analysis was shown to be 12 1/2 in. instead of 16 in. as obtained by the analysis used in the preliminary design.

The 33 Equations in Detail: The development of the equations is outlined to illustrate the procedure. There are 5 equations of the "bent" type such as:

$$M_{qt} + M_{tq} = -380W + D (380R_{qt})$$

where W is the external wind shear and D the direct dead and live load stress in the shaft.

There are 13 equations of the usual kind equating the summation of the moments at a point to zero. The summation of the moments at points k , o , and q , (Fig. 4), include the moments due to eccentric web connections.

There are 3 equations for the strains in members kt , ot , and qt . The development of these equations will be outlined later.

There are 4 equations for the deflection angles of shaft members between the chords of the 4 struts. These take the form

$$R_{be} = - \left[\frac{\Delta_{11}}{540} + \frac{0.000,000,724,034 T_1}{1080} \right]$$

in which Δ_{11} is the change in length of the shaft from the center of strut 1 to the base of the tower; the factor 0.000,000,724,034 is the vertical deflection, from a unit load, of the strut as a beam fixed at each end; and T_1 is the vertical shear carried by the strut. See Fig. 5.

There are 8 equations for the deflection angles of the two shaft sections, the two effective horizontal struts and the four diagonals in the fully braced section below the floor. The development of these equations is the most interesting feature of the study.

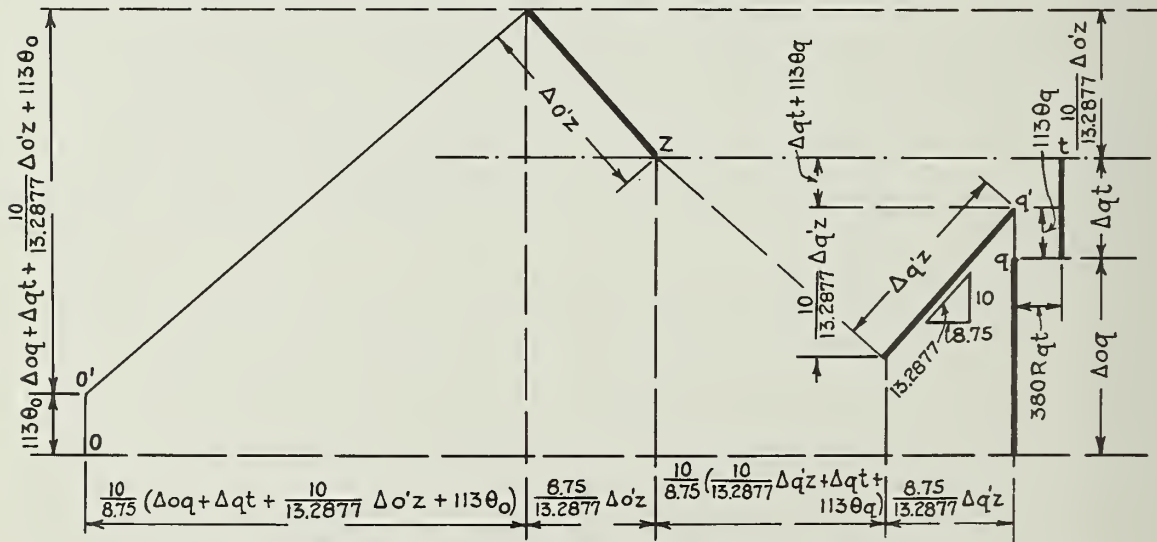


FIG. 6—Williot Diagram for bottom panel of bracing, typical of those used to obtain the deflection angle equations for the eight members below the floor.

First Williot diagrams were drawn for the movement with respect to the base of all points below the floor. One of these diagrams is shown in Fig. 6. From these Williot diagrams the deflection angles of these 8 members were expressed in terms of the strains in the members and the angles of rotation at points k , o and q .

A typical expression for the diagonals is Eq. 1 below. Eq. 2 gives a typical expression for the shaft members, while a typical expression for the horizontal struts is:

$$R_{oo} = -\frac{1}{540}\Delta_{ot}$$

This gives the set-up of the 33 equations finally used. However, equations for the stresses and strains in the members had to be developed and introduced in these equations before they could be solved.

The two horizontal struts at k and o were assumed to have no direct stress from wind. The typical expression for the stress in a diagonal is illustrated by Eq. 3.

The expressions for the stresses in the shaft members were first obtained in the form of the stress added at each point. A typical expression for a point below the floor is shown in Eq. 4.

Eq. 5 shows a typical expression for the load added to the shaft at a portal strut above the floor or the reaction from the strut (say strut 2).

The summation of these expressions for the load added at various points gave the total wind stress in the shaft members.

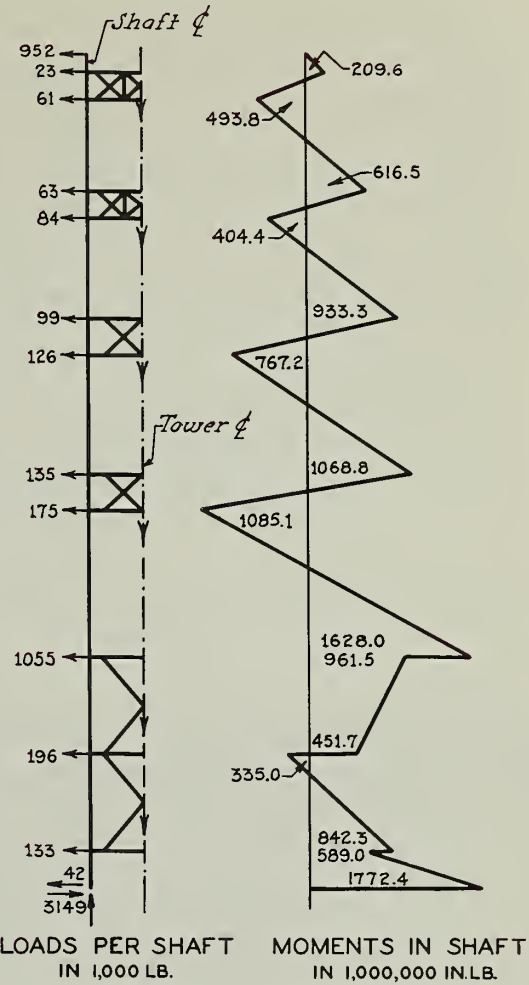


FIG. 7—Shaft Moments from transverse wind.

$$R_{q'z} = -\frac{13.2877}{8.75} \left[\frac{10}{13.2877} \Delta_{q'z} + \Delta_{qt} + 113\theta_q \right] \div \left[\frac{13.2877}{8.75} \times 427 \right] = - (0.7525757 \Delta_{q'z} + \Delta_{qt} + 113\theta_q) \div 427 \quad (1)$$

$$R_{oq} = - \left[\left(\frac{10}{8.75} \times \frac{10}{13.2877} + \frac{8.75}{13.2877} \right) (\Delta_{o'z} + \Delta_{q'z}) + \frac{20}{8.75} \Delta_{qt} + \frac{10}{8.75} \Delta_{oq} + \frac{10}{8.75} \times 113\theta_o + \frac{10}{8.75} \times 113\theta_q \right] \div 976 \quad (2)$$

$$= [1.32877 (\Delta_{o'z} + \Delta_{q'z}) + 2\Delta_{qt} + \Delta_{oq} + 113\theta_o + 113\theta_q] \div 854$$

$$\frac{8.75}{13.2877} S_{k'w} = 2778000 - 77413000 R_{ko} + \frac{M_{ko} + M_{ok}}{976} + \frac{10}{13.2877} \left(\frac{M_{k'w} + M_{wk'}}{648.439} \right) \quad (3)$$

$$\text{Stress added at } o = \frac{10}{13.2877} S_{o'w} + \frac{8.75}{13.2877} \left(\frac{M_{o'w} + M_{wo'}}{648.439} \right) + \frac{M_{oo}}{540} + \frac{10}{13.2877} S_{o'z} + \frac{8.75}{13.2877} \left(\frac{M_{o'z} + M_{zo'}}{648.439} \right) \quad (4)$$

$$\text{Reaction from strut 2} = \frac{-M_{de} - M_{ef} + 1104000 \times 264 - 64365000 \times 264 R_{de}}{540} \quad (5)$$

The strains were computed from these stresses and introduced into the 33 basic equations after the moments were expressed in terms of the slope deflection unknowns and the rotation and deflection angles of the various members. The 33 unknowns consisted of 30 angles and the strains in members ko , oq and qt which were kept as part of the final equations since they entered into several of the equations.

This method of analysis was so satisfactory that the time spent in earlier approximations was regretted. In any future design of this type this method of analysis would be used immediately after the first rough approximation by assuming a point of contraflexure midway between struts. The transverse wind loads and moments on each shaft as designed are shown in Fig. 7.

Other Stresses: Following the determination of the sections from dead and live loads, wind loads and longitudinal bending stresses, the towers were investigated for other stresses. In addition to the effect of the changes in temperature on the distribution of the suspended loads between the cables and the stiffening trusses and on the longitudinal position of the tower tops, consideration was given to the effect of a difference in expansion or contraction between the pier and the tower corresponding to a difference of 40° F. in temperature. The method of analysis was similar to that for the determination of stresses from transverse wind.

As a result of these studies it was decided that strut 7 should be made ineffective for carrying stress. The final stresses in the diagonals in the lower panel amounted to 1,300,000 lb. The maximum bending moment in the tower shaft was found to be 750,000,000 in.-lb. at the base of the tower.

The effect of the participation of the bracing below the floor in carrying the direct dead and live loads on the shafts was determined in a manner similar to that for the determination of temperature stresses, and the equations were nearly identical. The participation stresses in the diagonals averaged about 3,500,000 lb. The maximum bending moment in the tower shaft amounted to 725,000,000 in.-lb.



A study was made to determine the effect of unequal longitudinal deflection resulting from live load on one half the roadway. The primary purpose of this study was to determine the effect of such a condition on the lacing of the chords of the struts. The maximum difference in the deflection of the two shafts was found to be 8 in. Since there was uncertainty as to the torsional resistance of the cellular shafts stiffened by horizontal diaphragms, it was decided that the study should be made on the assumption that the shaft was as effective as a circular section of equal polar moment of inertia. Subsequent model studies indicated that the cellular shafts were about half as effective as was thus assumed.

It was found that twelve simultaneous equations could be written, the solution of which would give the loads on the shaft from the web members and the cable directly.

The longitudinal tower deflections at each of the twelve panel points for an unknown load at each panel point were determined. From these deflections were written simple expressions in terms of the unknown loads, for the deflection angles of the web members acting as horizontal beams.

The angles through which the shafts twisted at each panel point were found for a couple applied at each point. The couple at each panel point was equal to half the tower width times the unknown load except at the top where the couple was equal to the restraint of the cable or the horizontal component of the cable stress times the horizontal length of the effective part of the saddle multiplied by the unknown angle of twist. By equating this unknown angle to its value computed by applying the couples to the shaft, it was possible to eliminate this unknown angle of twist leaving only the twelve loads as unknowns. These torsional angles are also the angles of rotation of the horizontal beams comprising the web members.

From the expression for the deflection angles and rotation angles were written the expressions for the shears or end reactions of the horizontal beams which were equated to the unknown loads giving 11 of the 12 necessary simultaneous equations. The twelfth equation was found by equating the desired deflection at the top of the shaft less the deflection due to change in cable stress to the expression previously found for the tower deflection from loads at the panel points. A solution of these twelve simultaneous equations gave directly the shears in the struts due to the differences in the shaft deflections.

Although the process of determining the stresses appears somewhat elaborate for the determination of stresses that were realized to be subject to a considerable but probably fairly constant percentage of error, the investigation seemed justified since it revealed, quite accurately, the maximum possible stresses and the distribution of the stresses.

As a result of the investigation the three top struts were laced together in pairs reducing the resistance to torsion greatly. The lacing of the web members was increased as indicated by the results of these calculations.

It was realized that the secondary stresses in the members of the portal struts above the floor (Nos. 1, 2, 3, and 4) were relatively high and that the resistance of the

individual members to bending would contribute appreciably to the stiffness of the struts; that changes in length of the shaft between the chords of a strut would produce stresses in the strut members; and that there were other small stresses due to the change in slope of the deflected shafts in the distance between the chords of the struts.

It was desirable to keep the same rigidity of the struts that had been assumed when the transverse wind stresses were computed so as to avoid a recalculation of these stresses. At the same time, it was not wanted to waste material or permit excessive unit stresses in any member. Previous approximate calculations had indicated the probable amount of these stresses, and the sections in general were thought to be satisfactory.

Starting with the assumed sections, the secondary stresses were computed by moment distribution in the conventional manner. The stresses due to changes in length of the shaft were found by the ratio of the shaft strains to the deflection of the strut from equal and opposite vertical loads applied to the strut at the tower connections. The effect of the angular changes in the shaft was computed. The total unit stresses were computed. The summation of unit stresses indicated a few desirable changes in section and in the number of panels in the struts, and the calculations were revised to take care of these changes.

Following this part of the work which resulted in sections stressed under these assumptions to a reasonably uniform unit stress, the effect of the shears from secondary moments on the direct stresses was determined. This study indicated that the struts were slightly stiffer than had been assumed when making the calculations for transverse wind and the sections were slightly decreased to keep close to the original assumption for the rigidity of the struts. All the previous strut calculations were then revised for the new sections and the procedure continued until the assumed sections gave the proper rigidity.

The effect of the gusset plates on the stiffness of the members of the struts was included in the determination of the moments of inertia and areas of the various members. Since their effect was appreciable, it was realized that the rigidity of the struts as a whole could not be determined exactly. The procedure adopted gave accurate results within the limitations of the assumptions made. Previous studies indicated that a small variation in the stiffness of the struts had little influence on the stresses in the structure as a whole.

Earthquake Considerations: While the determination of stresses due to dead load, live load, wind and temperature presented a well defined problem subject to entirely satisfactory solution, it must be admitted that stresses due to seismic forces could not be evaluated with equal certainty. The towers, homogeneous, flexible shafts of steel, are anchored to massive concrete piers which are founded on rock. Although no one can predict just how a flexible shaft of this character will respond to an earthquake, some conclusions can be drawn as to its stability under these forces. In the judgment of engineers who have investigated destructive effects of earthquakes, these towers may



be subjected to earthquake vibrations of very small amplitude (a fraction of an inch) and accelerating forces amounting possibly to 5 per cent of gravity. In the completed structure the transverse deflection of the towers under the design wind load is more than ten times any expected movement of the pier tops, and the stresses from transverse wind will be more than double the stresses due to transverse earthquake forces. Due to the great flexibility of the towers in the longitudinal direction, stresses from longitudinal earthquake forces (5 per cent gravity) will not exceed 50 per cent of the longitudinal wind stresses.

The influence of earthquake forces on the design of the towers was a determining factor only in proportioning the anchorage to the piers so as to provide against earthquake while the towers were standing alone on the piers without the stabilizing effect of the cables and the suspended structure.

SOME DETAILS OF DESIGN: The gross area of each tower shaft at the base is 12,391 sq. in. and at the top 3,659 sq. in. As shown in Fig. 3, despite the fact that the four central transverse webs, to which the transverse bracing is directly connected, are made up of three widths of plates, they are, by splicing made as effective as single plates having a width equal to the full width of the shaft. Filler plates $\frac{1}{2}$ in. in thickness are used back of angles in locations where vertical joints occur in web plates. This makes it possible for tension plates to be inserted where it is desirable to carry tension across the joints without an offset or break in the main angles.

The tower shafts were fabricated in units which comprise "cell groups" of from three to six cells each in cross-section and generally about 40 ft. long. This grouping gave units within the working limits of major bridge shop facilities and also gave shipping pieces which were satisfactory in dimensions, weight and rigidity for handling. The largest units were about $7 \times 10\frac{1}{2}$ ft. in section, 45 ft. long and weighed 80 tons.

At horizontal splices the principal elements of the sections spliced, with few exceptions, are in direct bearing, and the splices are designed to develop 50 per cent of the compression strength in the sections spliced. An exception is that splices on extreme outer faces, both transverse and longitudinal, are designed to develop 60 per cent of the value of the section in compression.

Wherever the section of the tower shaft was reduced by omitting outer cells, a gradual transition from the larger to the smaller section was accomplished by lessening or tapering off, over a length of about 21 ft., the cross-sectional area of the cells omitted. This was considered necessary to avoid undesirable "hard spots" at these points and also to avoid overstress in the rivets at a point of abrupt change of cross-section.

Inasmuch as the transverse bracing connections are made directly to the four central transverse webs of the tower shafts, it is necessary to provide means for distributing to the other transverse webs their proportional part of the loads transferred at these connections. This was done through three planes of horizontal diaphragms placed in the tower shafts opposite each bracing connection. The diaphragms are tied together

by means of tension plates so that collectively they form an effective horizontal girder which is designed to carry to each transverse web of the shaft its share of the transferred load.

Manholes provided in the cell webs, to give access to all cells, are about 70 ft. apart and are staggered for safety. Those in the exterior faces of the shafts are completely covered, while those in the interior webs are left open and are fully reinforced to make good the section cut away. At frequent intervals, openings $6\frac{1}{2}$ in. in diameter were made in the web plates to facilitate delivery of field rivets to the interior cells.

Except for one cell in the center of each shaft, all cells are provided with horizontal diaphragms at intervals of from 12 to 15 ft. Each of these diaphragms is made up of one plate and four angles and is riveted to web plates on all four sides. The diaphragm plates were milled accurately to bear against the web plates so as to serve as spacers; the angles were set back $1/16$ in. A $1/16$ -in. filler was used between the angles and the web plates wherever these were joined by field rivets, as it was found that tight field rivets could not be obtained otherwise. These diaphragms were effective in obtaining accurate spacing of the webs and in maintaining the rectangular form of the cells.

Each tower is effectively anchored to the pier by riveted connections with pairs of angles imbedded for a depth of 52 ft. in the pier concrete. Also there are 78 dowels $6\frac{1}{2}$ in. in diameter imbedded in the pier concrete and projecting through the 5-in. base slabs, to form a positive shear connection between tower base and pier. The angle anchorage was arranged so that it could be riveted directly to the longitudinal web plates of the tower shafts while an initial tension was maintained in the angles by means of jacks.

In the completed structure the cable saddles are permanently centered in both longitudinal and transverse directions on the tops of the tower shafts. During erection, however, they were set slightly to shoreward so that, as load was applied to the cables, the saddles could be moved channelward (with respect to the tower-tops) and thus brought to final position by progressive stages. To facilitate this movement a nest of rollers 8 in. in diameter was placed under each saddle. As originally set the saddles were shoreward, on the San Francisco tower 5 ft. 6 in., and on the Marin tower 3 ft. 7 in. Temporary structural supports for the saddles in their "off center" position were arranged so that their movement could be controlled by means of hydraulic jacks. When the cables were strung and loaded and the saddles took their final position, they were bolted permanently to the towers and the spaces between the rollers filled with grout.

Owing to the magnitude of the towers and the unusual character of the design, it was thought desirable to have a model built for the purpose of checking its general behavior under the different conditions of load to which it would be subjected. The model was built of stainless steel on a scale of 1 to 56. Proportionate loads were applied to the model and unit stresses were measured by strain gauges. The results obtained from the model checked closely with the analytical results.

CABLES

The Bidding Plans and Specifications called for cables each composed of 27,572, No. 6, galvanized bridge wires, grouped in 61 strands of 452 wires each. The strands were to be arranged in the conventional manner which, when all in place and before compacting, would result in a hexagonal cross-section with sides at top and bottom. In the construction of other large bridge-cables, difficulty had been experienced in compacting the cables to a circular cross-section. Generally the horizontal diameter of the completed cable was longer than the vertical diameter and often this difference proved sufficient to give some concern in the fitting of cable bands. The contractor made a thorough analysis of the effect of strand arrangement on the shape of the cable and concluded that an arrangement which placed a vertex of the hexagon at the bottom instead of a side, would require a minimum vertical displacement of wires during compaction and result in a final cross-section conforming more closely to the ideal circular section desired.

This new arrangement of the strands which was adopted, carried with it some variation in the number of wires per strand with a view of getting them in the most advantageous position for compacting. While in the former arrangement, strands were

placed in the saddles in horizontal layers, in the new arrangement they were placed in vertical tiers. Fillers placed in the spaces between strands at the saddles were designed to hold the strands accurately in position at these control points.

The circular cross-section into which the cables are compacted obtains up to within 15 feet of the saddles at which points the spiral wrapping stops and retaining collars are placed. Between the retaining collars and the saddles the cross-section of the cables undergoes a transition from a circular shape at the former to the hexagonal shape which it takes throughout the length of the saddles. This length of cable in each instance is protected by means of a removable,

The vertical separators in a substantial frame kept the strands in proper relation with one another

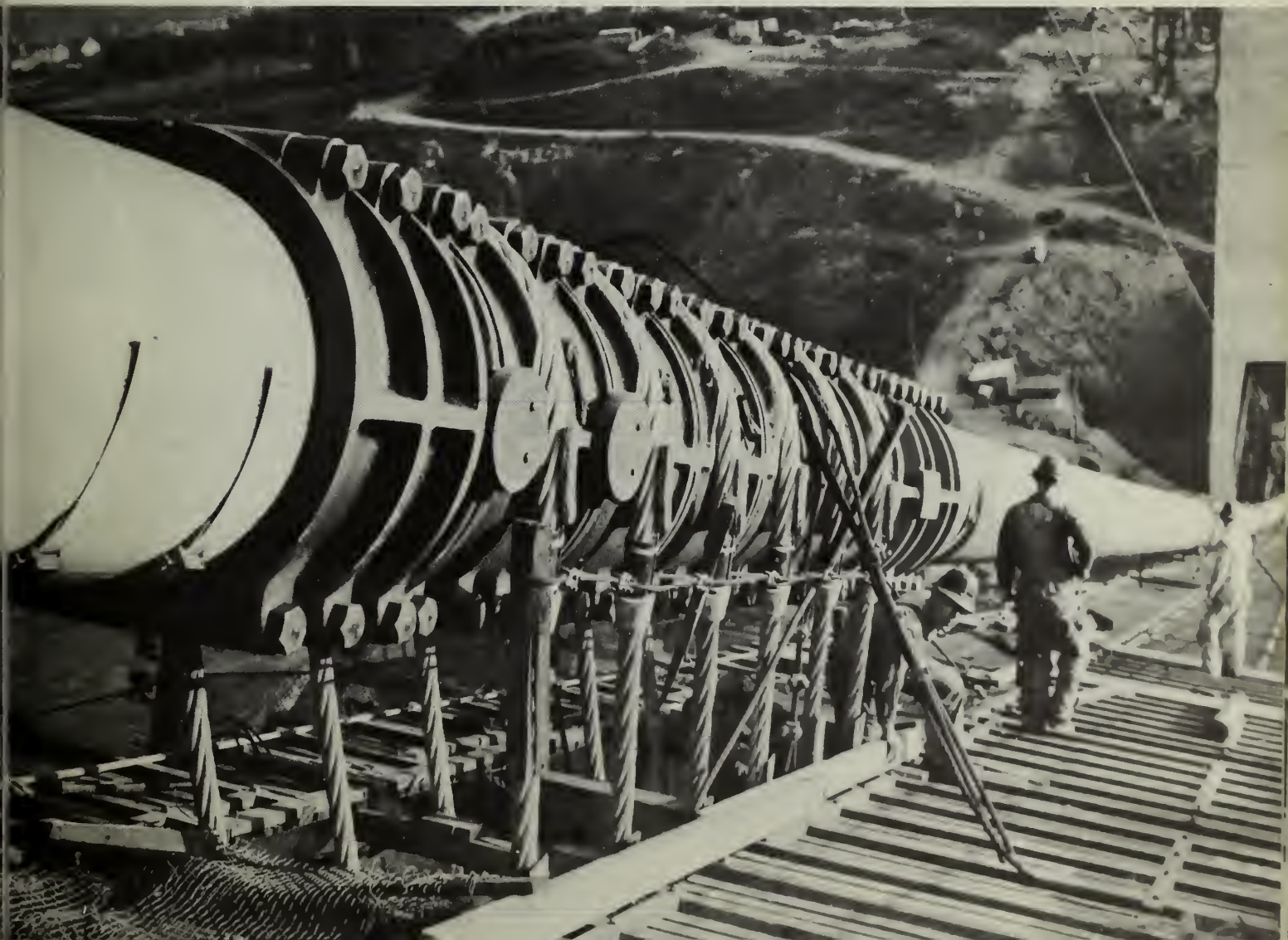


weatherproof, cylindrical enclosure supported by the retaining collar at one end and by a shroud casting bolted to the end face of the saddle, at the other end. The end connections of this cylindrical enclosure are provided with packing glands and are so detailed as to allow for all changes in direction of the cable due to changes in temperature and loads.

The cable groove throughout the length of the saddle has a weatherproof steel cover plate except through the central portion where it is surmounted by a small penthouse made of steel plates and angles. Manholes with weatherproof covers in the north and south faces of this penthouse, provide access for inspection of the cable within the saddle. Mounted on pedestals above the house of the two easterly saddles are the rotating airway-beacons required by the Department of Commerce.

At the shore ends of the sidespans, the cables are "tied down" to the Pylons by means of a group of short suspenders—six at each point. These, looping over an articulated cable-band, are adjusted to apply to the cable a load of 1,300,000 pounds under normal temperature and with no live load on the bridge. This load will increase to a maximum of 2,360,000 pounds under highest temperature and with full live load on center and far side spans only. By this means, the cable at that point is maintained at the proper elevation. Means are also provided at these "tie-down" points whereby the cables are there held against lateral movement.

The articulated cable band at one of the "tie-down" points



THE SUSPENDED STRUCTURE

The roadway and sidewalks of the bridge are directly carried by rolled-beam stringers which are seated on the top flanges of floorbeams. The latter occur at every panel point of the stiffening trusses which spaces them 25 feet apart. At every second panel point, the stiffening trusses are connected with the cables by double suspenders (4 parts) of wire rope 2 11/16 in. in diameter. The floor system and lateral bracing is in the plane of the top chords of the stiffening trusses. The stiffening trusses are 25 feet deep and their bottom chords, as explained elsewhere, are supported laterally by means of kneebraces at each floorbeam.

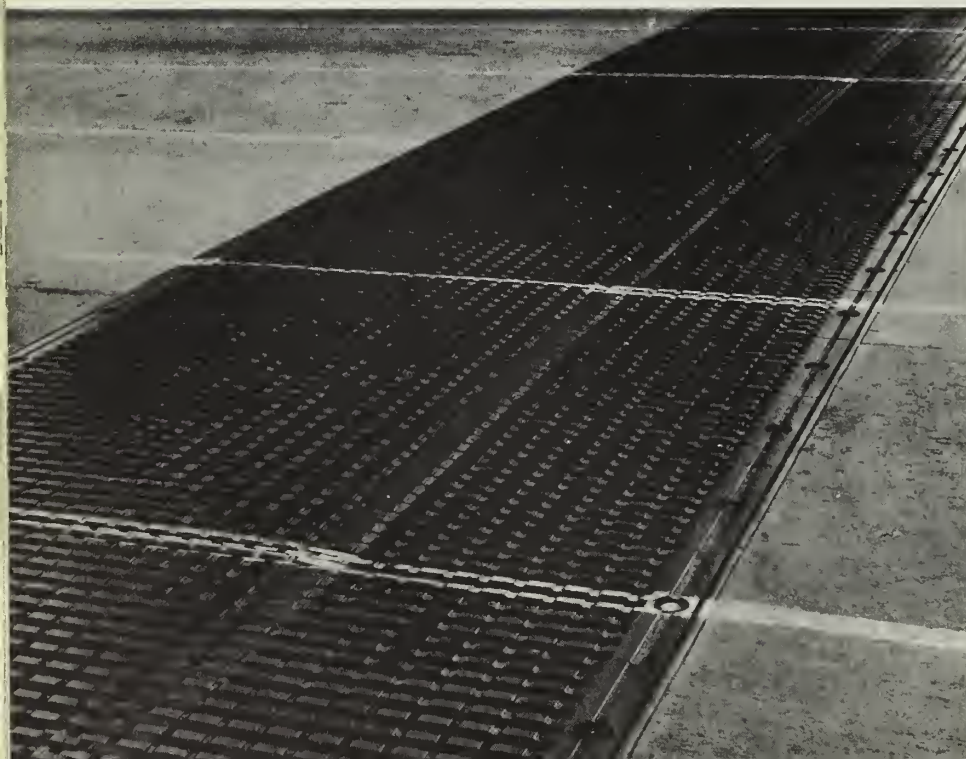
The floorbeams are 8 ft. 6 1/2 in. deep, 87 ft. 2 1/2 in. long and weigh 23 tons each. Webs are of carbon steel 1/2 in. thick and are spliced at two points. Flanges are of silicon steel and each consists of two angles 8x8x5/8 in. and two cover plates. The top-flange cover plates are both 20x1/2 in. The bottom-flange cover plates are one 20x1/2 in. and one 20x7/16 in. The floorbeams were cambered 3/4 in. and the ends were milled so that the end connection angles lie in vertical planes under full dead load.

The stiffening truss chords are of silicon steel. The diagonals and verticals are of carbon steel. Top chord sections have a maximum gross cross-sectional area of 145.51 sq. in. made up of two web plates 36x1 1/8 in., four angles 8x8x3/4 in. turned in, and one top cover plate 30x5/8 in. Bottom chords have a maximum gross cross-sectional area of 132 sq. in. made up of two web plates 36x15/16 in., four angles 8x8x3/4 in. turned in, and one top cover plate 30x5/8 in. In the side span stiffening trusses, the heaviest top chord section has a gross cross-sectional area of 91.23 sq. in. made up of two web plates 24x13/16 in., two top angles 6x4x5/8 in. turned in, two bottom angles 8x6x3/4 in. turned in, and one top cover plate 30x11/16 in. The heaviest bottom chord section has a gross cross-sectional area of 88.23 sq. in. made up the same as the heaviest top chord section except that the web plates are reduced to 3/4 in. thickness.

Riveted connections of stiffening truss members were designed to develop the full

strength of the members connected, at the allowable unit stresses used for their design. Diaphragms were used in the chord sections at every panel point and, in the case of the center span chords, at mid-panel points. Diagonals are box-sections formed of two ship-channels 18 in. x 51.9 lb. except for a few in panels near the end supports which are slightly heavier.

Roadway grating at expansion joint between tower and center span



Verticals are built up I-sections. Those at suspender connections are nominal members and consist of one web plate $32 \times \frac{3}{8}$ in. with $\frac{3}{8}$ in. reinforcing plate, and four angles $7 \times 3 \frac{1}{2} \times \frac{1}{2}$ in. Those at points intermediate the suspenders have to carry floorbeam loads and consist of one web plate $32 \times \frac{3}{4}$ in. and four angles $7 \times 3 \frac{1}{2} \times 9/16$ in.

The lateral bracing system is made up of box-members consisting of four or eight angles laced on four sides, designed to act either as tension or compression members. Since chord increments of wind stress are of considerable magnitude care was taken to apply these forces as near as possible to the neutral axes of the chords so as to minimize local bending.

Calculation of stresses in stiffening trusses was by the "deflection" method and the effect of distortion of long suspenders was included. This method has been so thoroughly described in connection with major suspension bridge design of late years that nothing need be added here. The stiffening trusses are exceedingly flexible and it will be of interest to note that unit stresses in the chords due to live load and temperature depend principally upon the chosen depth of trusses since great changes in the cross-sectional area of chord members would have very little effect on the cable deformation. The wind truss of the central span is likewise very flexible. Being more flexible than the loaded cables, there is a considerable transfer of wind load from the floor to the cables through the suspenders. At any suspender point the ratio of the transferred load to the suspender pull is equal to the ratio of the horizontal projection of the suspender to its length. Solution of simultaneous equations based on this relationship, gave the transferred load at each point. The calculated lateral deflection of the center span under a wind pressure of 30 pounds per sq. ft. is 27.7 feet and this is not much affected by changes in the cross-sectional area of the wind chords. The maximum downward deflection of the center span is 10.8 feet and its maximum upward deflection is 5.8 feet.

The combined unit stress in stiffening truss chords from live load, temperature and wind was therefore determined principally by the depth of the stiffening trusses and their distance apart. The use of silicon steel with an allowable unit stress of 32,000 pounds per sq. in. in tension and 25,000 pounds per sq. in. in compression for the severe combination of loads used in the design, met the requirements. The maximum chord stresses occur near the quarter points and of these the wind stress makes up about three-quarters of the total.

Wind velocities at the Golden Gate do not exceed 60 miles per hour and the bridge is not likely ever to experience lateral forces as high as those assumed. Unquestionably the bridge has ample stiffness in both vertical and horizontal directions. It is believed that the depth of the stiffening trusses could have been reduced considerably without detriment.

The wind truss of the center span is connected with the tower at each end through a hinged and sliding joint located on the longitudinal center line of the bridge. This connection is designed to transfer the lateral forces to the tower and at the same time permit free longitudinal motion as well as angular motion in both horizontal and



vertical planes. Stops are provided in this connection which limit the longitudinal movement towerward to 18 inches, and channelward to 21 inches, thus allowing a total range of movement of 39 inches. The calculated movements, exclusive of displacements due to longitudinal wind, is 12.5 inches and 16.5 inches respectively. At the curbs there is additional longitudinal motion due to the angular deflection which increases the movements there to a total of 26.5 inches towerward and 30 inches channelward. The maximum range of movement at the curb lines is therefore 56.5 inches and the grating joint in the roadway is accordingly designed for a closing of 27 inches from normal or an opening of 30 inches from normal.

The 25 ft. rocker links which support the center span stiffening trusses at the towers, swing through a considerable angle because of the change in length of the suspended structure. The upper link pin, as the link moves from its normal vertical position to one extreme or the other, describes the arc of a circle and causes the end of the suspended structure to drop appreciably. To accommodate this vertical movement the stringers of the end panel have one end hinged to those of the second panel and the other supported directly on the tower structure.

The wind truss of each side span is connected with the tower so as to permit only angular motion in horizontal and vertical planes. All longitudinal forces applied to the side span are delivered through these connections directly to the tower. The floor joints therefore are designed only to accommodate the longitudinal motion which results from the angular deflection of the side spans.

In the development of the stiffening truss plans it was necessary to consider the conditions which would obtain during various stages of erection since the geometric

figure of the truss would be greatly distorted and to accommodate this there would be considerable relative displacement of members at field connections. Studies were made to insure that a sufficient number of pins and bolts could be placed in the field connections, under such conditions, to take care of the erection and wind loads. Also special study was given to the closing of the stiffening trusses at the center of the main span where, at the time of closing, the cable would be $13\frac{1}{2}$ feet above its normal dead load position. This meant that the stiffening trusses would have to be connected up while under a deflection of $13\frac{1}{2}$ feet upward. Because of this deflection, with the closing top chord member pinned in place, the opening left for the closing bottom chord would be about 3 inches less than the normal panel length. Accordingly the milled length of this member was made $3\frac{7}{8}$ inches short so that it could be entered, after which, with the aid of a 350 ton jack, the panel length would be increased to normal and the connection made. This would give to the stiffening truss throughout its length, the configuration which it should have. It will be noted that, except for the closing bottom chord mentioned, all chord members were milled to bear and the only open joint is in the closing panel at the center of the main span.

The center span rocker links are subject to considerable torsional deflection resulting from angular deflection of the suspended structure under transverse wind load. In order to avoid excessive unit stresses from this torsion, each link was designed as a pair of built up I-sections joined together by diaphragms at top and bottom only.

The roadway slab, carried on stringers spaced 4 ft. 9 in. apart, is seven inches deep. It is reinforced with fabricated reinforcing trusses having an overall depth of $4\frac{5}{8}$ in. and spaced six inches apart. Top and bottom chords $11/16$ in. diameter are arc-welded to webs $7/16$ in. diameter in the form of a Warren truss. The bottom of the truss was spaced 1 in. above the top of the stringer by means of short, steel lugs which were arc-welded to both the stringer and the truss. One inch cover of concrete was provided for the bottom chords which allowed $1\frac{3}{8}$ in. cover for the top chords. Longitudinal rods $\frac{3}{4}$ in. diameter were placed in the bottom of the slab, four in each panel between stringers. Longitudinal rods $\frac{1}{2}$ in. diameter were placed in the top of the slab, one over each stringer and four in between. Concrete was specified to have a strength of 4000 lbs. per sq. in. in 28 days.

No light-weight concrete was used

The handrailing at the ends of the suspended spans is joined with the handrailing around the tower by means of a telescoping, hinged section of railing which can accommodate itself to the $5\frac{1}{2}$ ft. range of movement of span with respect to tower. The walkway around the tower is one step above the walkway on the span so that the latter is free to move underneath the former





except for the west sidewalk where encasement of conduits increased the volume of concrete and in order to keep the weight per foot the same as that on the east side, a concrete weighing approximately 100 lb. per cu. ft. was specified.

The roadway and sidewalk pavement is made in units fifty feet long with a $\frac{3}{4}$ in. transverse opening between units. These openings are faced with steel plates which support an inverted U-shaped, copper seal and a shallow pre-molded bituminous filler at the top of the slab. These joints prevent the pavement from participating in the strains which the top chords of the stiffening trusses must undergo. For construction purposes two longitudinal construction joints were provided in the roadway slab which divided it into three strips each approximately 20 ft. wide.

APPROACHES

The Marin approach proper comprises a series of five 175-foot deck truss spans supported on high, steel towers. This structure, having an overall length of 1100 feet, carries the roadway from the Marin Anchorage to the beginning of the state highway approach road which links the bridge with Route 101 at Waldo Point. With a curve of 1025 feet radius, it swings over a deep valley requiring towers of varying height up to 160 feet. Because of their height the towers were kept rectangular in plan and the curvature in the structure was effected by skewing the ends of the spans. This avoided a large number of bent plates for bracing connection of towers.

On the San Francisco side, that portion of the structure which extends from the Shore-end Pylon at Fort Point southward over the fort and the San Francisco Anchorage to an abutment about 170 feet north of the Toll Plaza has been termed the "San Francisco Approach" and has an overall length of 1072 feet. The major structure of the approach is a two-hinged arch span over the fort having a length of 319 ft. 7 in. center to center of bearings. The arch has four ribs. The two inner ribs are spaced 42 ft. 6 in. apart and the two outer ribs are spaced 30 ft. outside the inner ribs. The arch rise is 113 feet and, at the north end, the roadway is 185 feet above the springing line. The outer ribs being spaced 102 ft. 6 in. apart puts them 6 ft. 3 in. outside the cable backstays.

At the south end of the arch span a concrete pylon marks the north end of the anchorage housing and rises above the roadway with an architectural treatment similar to that of the Shore-end Pylon. Southward of this pylon are two 125-foot deck truss spans supported on two steel towers which rise from the anchorage housing. From the southernmost tower on the anchorage, to the abutment, the structure comprises first a 175-foot deck span and then three 71-foot plate girder spans.

The deck of the San Francisco Approach is similar to that of the suspended structure employing steel curbs and steel handrailings. In the case of the Marin Approach however, the concrete curbs and balustrades necessary over the Marin Anchorage, are continued for the full length of that approach.

The main span stiffening trusses and floor were extended outward in both directions from each tower. On November 18, 1936, the two main-span travelers met at mid-span and the stiffening trusses were joined together. At this time the main span was deflected upward $13\frac{1}{2}$ feet above normal

The Presidio Approach Road which joins the Toll Plaza with Marina Boulevard at the eastern or Lyon Street boundary of the Presidio has two structures which for convenience have been named the High Viaduct, and the Low Viaduct. The former has an overall length of 1518 ft. 9 in. and comprises eight 135-foot deck truss spans flanked on each end by four beam spans. The Low Viaduct has an overall length of 3308 ft. and consists of a reinforced concrete viaduct having span lengths of approximately 30 feet. Each bent comprises two columns with a cross girder and one or more horizontal struts depending upon its height. Bents are joined together in groups of two, to form towers. The roadway slab and stringers for each tower are poured monolithic with the upper column sections and cross girders, and one or more longitudinal struts are provided depending upon the height of the bents. The spans between towers are fixed at one end, and at the other end are provided with steel bearing plates designed to slide on bronze bearing plates set on the supporting brackets.

Both viaduct structures carry a 60 ft. roadway and one 3½ ft. sidewalk. Curbs and balustrades are of concrete similar to those on the Marin Approach.

The approach from Richardson Avenue passing southward of the Palace of Fine Arts joins the Low Viaduct by means of two ramps. This approach provides three lanes for off-going traffic and three lanes for on-coming traffic. The latter pass under the Low Viaduct and thence up a ramp to join traffic bound toward the bridge from Marina Boulevard.

Present construction anticipated the requirements of a future Funston Avenue link and made provision for a braided connection whereby on-coming traffic lanes from Funston Avenue would pass underneath the Presidio Approach Road before joining it just westward of the west abutment of the High Viaduct.

TOLL PLAZA

Obviously the toll collection facilities should be located on the San Francisco side, and the Board of Directors of the District decided that the District's Administration Building should adjoin the toll plaza. The area finally granted the District by the Presidio authorities for this purpose is 350 ft. wide and 500 ft. long and, although the 500 ft. length is less than that requested and desired to permit easy approach to the outside toll lanes, an operative layout has been effected. An increase of 150 ft. in length would have made a great improvement.

Fourteen toll lanes were deemed necessary to adequately handle all the traffic which the bridge could accommodate. In view of the long approaches to the Toll Plaza, which from the boundary lines of the Presidio are in no way connected with other Presidio roadways, it seemed desirable to provide an underpass below the plaza by which cars, entering the plaza by mistake, could turn back without crossing bridge traffic. Other frequent users of this underpass would include District officials and employees as well as tradesmen and others who would have occasion to turn around at the plaza. Aside



from toll booths, housing was required for administrative offices, shops, garages, store-room and powerhouse, as well as quarters for the State Highway Patrol stationed at the bridge.

Several layouts were made, each meeting the above requirements but differing principally in the size, arrangement and type of construction of the administration building and shops. The plan adopted placed the administration offices, shops, storage, and power house all in one building on the west side of the plaza. On the east side, in line with the toll booths, the quarters for Highway patrolmen were placed. This arrangement was the least costly and although it lacks symmetry that objection is not prominent.

Portions of the plaza site required a deep fill. However, all structures are carried on foundations to rock. The underpass structure itself carries the roadway above it, as well as the toll booths and scale pits. Anticipating future settlement of the fill, mud jack sleeves were specified to be included in the pavement slabs, centered 5 ft. 6 in. in one direction and 7 ft. in the other direction.

Each toll booth is provided with a key box which has a separate key for each classification of traffic. When the toll collector presses a key corresponding to the classification of the vehicle for which he is collecting the toll, automatically the amount of the toll is indicated to the driver; the classification number is lighted on the overhead indicator above the lane; and the transaction is registered remotely. Axle counters in each lane remotely register the number of axles passing through the lane and this gives a check against the number computed from the registration made by the toll collector. Each collector is provided with a distinctive, identification key which he inserts in the key box at the beginning of his shift and removes whenever he leaves the booth. At each insertion and removal of the key a printed registration of the date and time of day, and the axle count, is made at the remote register. The scales are equipped to automatically register remotely the amount of the toll collected from trucks according to weight. Receipts at the same time are automatically printed showing the amount collected according to weight.

POWER, LIGHTING AND SIGNAL FACILITIES

To meet government requirements it was necessary for the District to provide, maintain and operate navigation lights and signals comprising: special aid to navigation light for regular U. S. Lighthouse service on the San Francisco Pier; revolving airway beacons on each main tower; pier lamps on both main piers; mid-channel lights—6 white and 2 green; 6 obstruction lights on each cable; two diaphones at the center of the main span; and two typhons at the San Francisco Pier. These navigation lights and signals must be ready to function at all times and therefore it was necessary to provide an emergency power source to supply electric energy should there be an interruption in the normal power service. Also a standby air-compressor unit was necessary to

maintain air pressure while the other unit might be shut down for repairs. These were the principal needs to be met aside from roadway lighting, building lighting, intercommunicating telephones, and power elevators.

When the bidding plans of 1931 were prepared, the conception of electrical needs was not complete. At that time the plans provided for a substation below the bridge floor at the San Francisco Tower where the District would receive power on the secondary side of the 11,000/2300 volt transformers which would be furnished by the power company. Here also would be located the motor driven air-compressors and the emergency power unit. As plans for the project matured, it became obvious that insofar as practicable, all electrical and mechanical equipment should be centralized in one building at the Toll Plaza. Accordingly new plans were made which accomplished the aforesaid centralization and also provided for the sodium vapor roadway lighting that had been decided upon as early as 1933.

The electric service, in the form of alternating current, 3 phase, 60 cycle, 11,000 volts, is delivered to the main control panel in the Power House at the Toll Plaza. Here it is stepped down to 2300 volts by means of transformers of which duplicate sets are installed. One set is to furnish "preferred" service and the other "emergency." By means of a selector-switch either set can be made to function as "preferred" and the other as "emergency." Either set of transformers is capable of taking the entire load and if for any reason the selected "preferred" set becomes inoperative, the "emergency" set will automatically take the load after a short time-interval.

Should the 11,000 volt service be interrupted for any reason, a gasoline engine generator-set will be automatically brought up to voltage and thrown on the 2300 volt bus. The arrangement is such that when this is done the main service will have been disconnected from the bus, it being impossible to throw the gasoline engine generator-set on the bus as long as the 11,000 volt service is connected to it.

When the 11,000 volt service is restored, the gasoline engine generator-set will be automatically disconnected from the bus after a time-delay, and the 11,000 volt service will be automatically thrown on the bus.

As an exception to the above, it has been arranged so that during daylight hours, when no lights are being used, the automatic starting of the generator-set will not take place unless the air pressure is low.

When the gasoline engine generator-set is supplying the electric energy, no current is available for the roadway lighting circuits, but current is available for the balance of the equipment.

The series lighting circuits, seven in number, are controlled from a supervisory switchboard where each circuit can be independently switched on or off at the will of the operator.

Roadway lights on the Suspended Structure and the Marin and San Francisco Approaches are spaced 150 feet apart on each side of the roadway and are set opposite one another. The lighting circuits are 2300 volts, 6.6 amperes, constant current

Lighting units are General Electric Company Sodium-Vapor Luminaires, each provided with a 6.6 ampere IL transformer for a 10,000 lumen, series lamp.

Two motor-operated air-compressors have been provided. Each has capacity to meet all requirements. Either can be selected as the "preferred" unit, and the other as the "emergency" unit. Ordinarily one compressor is in service, and it automatically starts and stops at predetermined air pressures. If, however, this compressor fails to start, the emergency compressor-set will automatically start when the air pressure drops 5 pounds below the normal starting pressure.

In the east shaft of each main tower there has been provided a small, electric service-elevator which gives access to the tower base, the tower top and to each intermediate strut. These are intended particularly to facilitate servicing of pier lights at the tower-bases, and airway beacons at the tower-tops.

A storage battery has been provided to furnish current for starting the gasoline engine; for the indicator lights; for operating the oil switch; and for emergency lighting of certain lamps in the power house and at the toll booths.

BOOK THREE



BOOK THREE

CONSTRUCTION INVOLVED NEW PROBLEMS FACED AND MET WITH COURAGE AND RESOURCEFULNESS BY THE COMBINED FORCES OF THE ENGINEERING STAFF AND BUILDERS.

CONSTRUCTION

BEFORE the contracts for construction were drawn a careful study of the construction schedule was made to determine the most economical program for the District. This schedule took into account savings that might follow by avoiding duplication of expensive contractors' equipment and keeping interest during construction at a minimum. Interest cost during construction was kept at a minimum not only by building each unit of the work as rapidly as possible, but also by scheduling the completion of each unit so that it would be ready simultaneously with the other units required for the next stage of construction.

The construction schedule of a suspension bridge is narrowly defined by the natural sequence in which the various units must be built. A control point in the sequence is the start of cable erection. It is necessary at this point that both the main towers and the cable anchorages be completed ready for the cable erector. If the completion of these various units is simultaneous and if their construction has been accomplished in the minimum time, then the interest cost has been kept at a minimum. The construction of the various foundations, the anchorages and piers are tasks of different magnitude and the proper scheduling of the construction of these foundations determines to a large degree the economics of the entire construction schedule. Included in the work to be done before the critical date of starting cable erection, is the construction of the two main towers which, if carried on simultaneously, involves a duplication of expensive equipment and contractor's organization. A saving in the contract cost of the main towers is effected when the same equipment and organization can be used on both towers.

On the Golden Gate Bridge, the construction of the two main piers presented tasks so greatly varied in magnitude that the District had the option of either constructing the Marin Pier and Tower while the longer task of building the San Francisco Pier was under way and thus permit erection of the San Francisco Tower later with the same equipment and organization as the Marin Tower, or not starting the Marin Pier until later and then erecting the two towers simultaneously. The Engineer chose the former program. This program was obviously to the best interests of the District since this scheduled the work in accordance with the capacities of fabrication plants of either of several bidders and permitted the District to take advantage of the duplication in fabrication and save by avoiding duplication of equipment for erection.

The following schedule was therefore in force at the start of the work. It was incorporated in all contracts not only for the main bridge, but for the approaches so that all work was contingent upon the main schedule for completion.

Sept. 1, 1933—The Marin Pier to be completed.

April 1, 1934—The San Francisco Pier and Fender to be completed.

Oct. 1, 1934—The Anchorage Base Blocks and the Anchor Blocks, including the Anchorage Steelwork to be completed in readiness to receive the cables.

Jan. 1, 1935—The San Francisco and Marin Steel Towers to be completed in readiness to receive the cables.

Jan. 1, 1936—Cables (except wrapping) to be completed.

June 1, 1936—Anchorage Weight Block to be completed.

June 1, 1936—Stiffening Trusses and Floor Steel to be completed.

Dec. 1, 1936—Concrete Pavement to be completed.

Work on all four main foundations viz: main piers and anchorages was begun simultaneously. First to be completed was the Marin Pier.

CONCRETE MATERIALS AND HANDLING

One of the unique features in connection with the contracts for the anchorages and main piers was the arrangement for the delivery of ready-mix concrete to the point of deposit. These two contractors made an agreement with the Pacific Coast Aggregates Co., a building materials firm, whereby Barrett & Hilp, contractors for the anchorages, built a dock on the Marin side of the Gate and the Pacific Bridge Co., contractors for the main piers, built a similar dock on the San Francisco side. The Pacific Coast Aggregates Co. installed equipment on both docks to comprise proportioning or Batching Plants with all the latest improvements. These proportioning plants being located close to the work supplied practically all of the concrete for the project. The San Francisco Batching Plant was located one-half mile from Fort Point which was the nearest point where barges could be anchored safely the year around.

On the San Francisco side, aggregate was brought to the plant on barges having a capacity of 500 tons. Barges were unloaded with a Wiley Whirley using a clam-shell bucket of three-yard capacity at a rate of about 160 tons per hour. Aggregate was placed in three storage bunkers, one for sand and two for gravel, each bunker having a capacity of 450 tons.

The aggregate was conveyed by belt from the storage bunkers to two bins, one of which was located above each batcher. Each bin was divided into four equal compartments and had a total capacity of 200 tons.

Cement was brought to the material dock in bulk on barges carrying 2000 barrels each. A Fuller-Kinyon portable pump with a capacity of 100 barrels per hour was used to unload the cement, pumping the cement into a 500-barrel tank mounted over a stationary pump. This pump discharged the cement through a 5-inch pipe line from the 500-barrel tank into two storage tanks, each with a capacity of 1500 barrels and mounted adjacent to each batching unit.

On the Marin side, aggregate was unloaded from barges with a steam locomotive crane. There were twelve bunkers each with a capacity of 100 tons. The cement was

pumped from barges with a portable Fuller-Kinyon pump into one 2000-barrel capacity tank. A stationary pump conveyed the cement through a pipe line into two tanks having a capacity of 250 barrels each and mounted adjacent to each batching unit.

Cement, aggregate and water were measured separately and batched directly into truck mixers. Fairbanks visible dial scales were used for weighing aggregate and cement. The aggregate scale was separate from the cement scale and had four beams, each with a capacity of 6000 pounds. A poise was set on each beam for the required weight, the dial indicating that weight, and as the material was discharged into the weighing bin the dial pointer approached zero. When zero had been reached, the correct weight of material was in the weighing bin and the operator manually closed the gate controlling the flow of that particular aggregate. A flag mounted on the dial indicator intercepted a photo-electric beam at zero, actuating a relay and causing a solenoid to unlock the next material gate and allowing the next scale beam to be used.

The cement was weighed by a separate scale into a separate weighing bin. The poise was set on the beam for the required weight and this weight indicated on the dial. A screw conveyor carried the cement from the storage tank into the cement weighing bin. In operating the device, when the dial pointer reached zero, a photo-electric beam was intercepted thereby automatically shutting off the electric motor driving the screw-conveyor and stopping the flow of cement.

When the aggregate and cement had been weighed into their respective weighing bins, a discharge gate was automatically unlocked, permitting the batch to be discharged into the truck mixer. The opening of the discharge gate on the weighing bin automatically actuated the discharge of water from a syphon-type Ransome tank. The

The Marin concrete
batching plant at
the right



water discharge was controlled by a calibrated wheel, the setting of which governed the point at which the syphon broke, stopping the flow of water at the predetermined point.

Each batching plant was equipped with two separate batching units. All discharge gates were lever-operated manually. With both units operating, the plant readily turned out 120 cubic yards per hour.

The concrete was mixed in Mixer-Trucks. The unit comprised a mixer-drum mounted on a truck chassis and powered through a power take-off from the truck motor. A hydraulic hoist raised the mixer for discharging the load. The mixer-drum was cylindrical in shape and had a frustrum of a right circular cone at one end for charging and discharging. The overall length of a mixer-drum was 10 feet 8 inches and the mixing capacity four cubic yards. The mixer-drums were equipped with pick-up and overthrow blades and their mixing speed was 15 R.P.M. A hinged gate with rubber hose gasket was fastened at the end opening.

Each mixer truck was provided with a timing device or meter-box for the purpose of controlling the mixing time of each load. This device was so arranged that the load ticket was perforated while passing through a slot in the meter-box at a fixed speed. The time required for the ticket to pass through the slot was equal to the required mixing time. The ticket was inserted in the slot at the time the load was completely charged into the mixer and the load was not discharged until the ticket had passed entirely through the slot and was perforated for its entire length. The meter-box was connected by a flexible speedometer cable to a friction plate on the mixer-drum shaft. The speed of the mixer-drum regulated the engagement of a clutch in the meter-box. This clutch would engage only at speeds of 14 R.P.M. or over, thereby causing the ticket to move only when the mixer was turning within the required speed range.

Aggregate samples were secured from barge, belt and bin with a scoop or shovel. All barge loads were sampled and in such manner that a composite sample was made from several smaller samples taken from various parts of the barge. Composite samples weighed about 40 pounds for coarse aggregate and about 20 pounds for fine aggregate. The latter samples were quartered down to about 500 grams for testing. Belt samples were occasionally taken as a check on barge samples.

Moisture contents of the aggregates were determined from samples secured at the batching plant bin. These moisture determinations were made daily and several times daily during large pours.

A field testing laboratory was established at each plant. The necessary equipment was installed for the testing of aggregate and cement and for experimental and test work in concrete. These laboratories were the control points for all concrete production.

Concrete mix designs were based on the following theories:

1. Water-cement ratio for strength, the volume of water per sack of cement having a direct effect upon the strength of the concrete obtained.
2. Fineness modulus grading, the grading of the individual and combined aggregates in the mix, determining the consistency and workability of the mix.

3. Absolute volume, the yield of a mix being determined by the sum of the absolute volumes of the component parts in a mix, including water and cement.

Physical characteristics and specific-gravity tests were made on all fine and coarse aggregates used, in accordance with the latest approved A.S.T.M. Standards, from which weight-conversions were made, batching of concrete being done by weight instead of volume. Cement testing was done by Smith, Emery & Co., a commercial testing laboratory.

All mixes, with the exception of a few "lean" weight mixes, were designed upon a minimum strength requirement at 28 days, tests on the set concrete being made according to A.S.T.M. Standards. In addition, a minimum cement content of 5.2 sacks per cubic yard of concrete was specified. Cement contents of the various strength concrete mixes used ranged from 5.2 to 8.0 sacks per cubic yard, with grout mixes from 8.0 to 12.0 sacks. Strength requirements varied from 2500 pounds to 4000 pounds per square inch, and the maximum size of aggregate varied from $\frac{3}{4}$ inch to $2\frac{1}{2}$ inch (square hole) according to particular requirements of placing. Grouts for topping fillers under base plates, etc., were limited to $\frac{1}{4}$ inch maximum. In all, about 50 basic mixes were designed and used, with variations in each mix due to changes in aggregate grading, moisture, etc.

Upon notification of the starting of a concrete pour, the batching-plant inspector set the weighing scales for the predetermined mix, checking moisture contents before setting water wheel. After batching the first load into the mixer-truck, the truck was held at the loading ramp, and the load thoroughly mixed for four to five minutes, then examined as to consistency and slump. If necessary, corrections were made for the following batch, and this batch mixed and examined. Further control of the concrete was exercised by the batching-plant inspector through his inspection of concrete at placement and his contact with the placing inspector. As placing proceeded, the placing inspector would notify the batching-plant inspector by note or telephone as to any changes thought necessary in the consistency or workability of the concrete. Additional control was furnished by frequent slump tests made both at the batching plant and at the pouring.

Concrete test cylinders were made during each pour, one 7 day and one 28 day cylinder being made. On large pours, test cylinders were made every two hundred yards. After a concrete mix had been used consistently, and a good idea of the quality and strength of the concrete had been obtained, the seven day tests were omitted and only the twenty-eight day tests made. Specifications required 8 inch by 16 inch test cylinders, but due to the insufficient load capacity of the testing machine, 6 inch by 12 inch test cylinders were substituted. The molds used were either 6 in. by 12 in. steel tubing fitted with spring clamps and $\frac{1}{2}$ in. thick steel base plate, or the standard 6 in. by 12 in. paraffine paper mold with tin bottom.

Test cylinders were made as concrete was being placed and at the site of placing, to minimize handling of wet concrete from site to field laboratory. A truck load of concrete was half emptied before securing the test sample, in order to approximate average

conditions in the batch of concrete. Test cylinders were made by placing the concrete in the mold in layers 3 in. to 4 in. thick and rodding each layer 25 times with a bullet pointed $\frac{5}{8}$ in. diameter rod. After rodding the top layer, the surface concrete was leveled off, covered, and the cylinder placed under cover in a location free from vibration or movement to allow the fresh concrete to obtain its proper set. After allowing to set for 48 hours, the molds were stripped off, the cylinders marked and taken to the testing laboratory, care being taken not to injure them in transit.

On arrival at the laboratory, the test cylinders were placed in a moist closet, heated to 70° . When a sufficient number were on hand, they were removed from closet to an adjoining level table, placed in a vertical position and capped with a mixture of plaster of paris and cement. Oiled glass plates were used to obtain a true level surface, being pressed upon the fresh capping cement, leveled off with a hand level, and excess material was removed from under the plate by a spatula. After the cap had hardened, the cylinder was reversed with the plate still in position, and the opposite end capped. Upon completion of capping, cylinders were replaced in a moist closet to prevent cracking and shrinking of caps, the glass plates being removed the following day.

Cylinders to be tested were removed from the moist closet, wiped off, weighed, average diameter and height measured, ends checked for levelness and placed in a 200,000 pound Riehle vertical hydraulic compression machine. An adjustable bearing block was used on the top end to compensate for any inclination of the cylinder from the vertical. Load was applied to the cylinder uniformly and without shock.

The total load indicated by the testing machine at failure was recorded, and the unit compressive strength calculated in pounds per square inch of area. The type of failure and appearance of concrete were also noted. All data obtained was entered upon the laboratory test sheet made for each cylinder, identifying the cylinder with information concerning mix, slump, location, age, etc.

ANCHORAGE CONSTRUCTION

The construction of the main anchorages and pylons, together with the piers and abutments for both the Marin and San Francisco Approaches was undertaken under Contract III by Barrett & Hilp. Although the size of the two anchorages varies slightly on account of the difference in topography, each consists of the same three distinct types of construction. The anchorages proper are of heavy mass concrete requiring methods similar to dam construction; the housings are light reinforced concrete buildings; and the pylons high, hollow, reinforced concrete shafts.

Construction of the anchorages was divided into two stages. The first stage included not only those parts necessary to permit the erection of the main cables, but also as much of the housing and pylons as could be built without interfering with the operations of the cable erector. This was done in order to minimize the amount of work that had to be performed after completion of the cables.

The anchorage at either end of the bridge comprises two separate masses of concrete into which each of the cables is anchored. Each mass was constructed in three units: the base block or foundation; the anchor block containing the anchor chains and girders; and the weight block which rests on top of the anchor block.

The Marin Anchorage is located against the side of a sandstone and shale hill. The bottoms of the East and West base blocks are at elevations 78 and 84.5 respectively. The San Francisco Anchorage was founded deep in the serpentine ledge along the shore at elevations -35 and -45 for the East and West base blocks respectively.

The bottoms of the excavations together with the faces toward the bridge were so cut into the rock that the bearing surfaces would be normal to the reaction of the completed structure. This involved cutting the bottom of the excavation into a toothlike surface with the depth of the teeth approximately four feet.

On the Marin Anchorage, excavation was done by means of a power shovel and blasting powder. Suitable rock for the foundation was uncovered at plan-depth except for one corner of the east base block where a small amount of additional excavation was required.

On the San Francisco side, the contractor elected to first relocate and rebuild an old granite seawall out a sufficient distance from the structure so that it could be used as a cofferdam. Construction of this seawall was carried on at low tide and its footings placed in the dry.

The Marin Anchorage
—eye-bar chains in
process of erection



During all of the excavations for the base blocks no pumping was necessary. The material for the San Francisco Anchorage was excavated with a power shovel and blasting powder. Careful watch was kept on the drilling and loading of holes for the blasting to prevent overbreak that would have impaired the solidity of the foundation bed. The serpentine formation in which the excavation was located was sound and impervious. The bottom of the pit was dry at all times. Spoil from the excavation was deposited outside the newly located seawall.

The blocks were divided into pours of such size that a layer of concrete could be placed over fresh concrete that had not yet reached the initial set. These pours were laid out so that maximum resistance to shear would obtain under the stresses existing in the completed structure. Because of the extreme width

of the blocks (sixty feet,) the ordinary methods of bracing the forms by the use of hog rods and struts were not convenient. The contractor developed an ingenious scheme whereby, as the concreting proceeded, one foot square, steel anchor plates were embedded in the fresh concrete. To these plates were attached $\frac{5}{8}$ -inch diameter rods which projected toward the forms at an angle of approximately 45 degrees. As the concreting progressed, the anchor plates were sufficiently restrained by the initial setting of the concrete so that the walers could be fastened to the rods and held to true lines.

After the base blocks had been completed, one pour was made on the anchor blocks so that the erection of the anchor chains and girders could be begun. The latter material was furnished under Contract I-A and erected by the contractors for the anchorages. The backs of the anchor girders rested against a cylindrical surface with the axis of the cylinder at a slope of .29594 and .31711 on 1.0000 from the vertical in San Francisco and Marin respectively. The center planes of the girders were set radial to this cylindrical surface. The anchor chains were supported on steel falsework which was embedded in the concrete of the anchor block as concreting progressed. Special methods were used to facilitate the full distribution of the cable load to the anchor girders. As an aid to this end, the heads of the eye-bars and all projecting steelwork ahead of the anchor girders were cushioned with expansion joint material. In addition, the steelwork was



The Marin Anchorage
—eye-bar chains nearly
erected



Above: The Marin Anchorage—Below: The San Francisco Anchorage



given two field coats of paint which may be of assistance in preventing bond between the concrete and the eye-bars although it is believed that a coating for the bars could be found that would be more effective for this purpose.

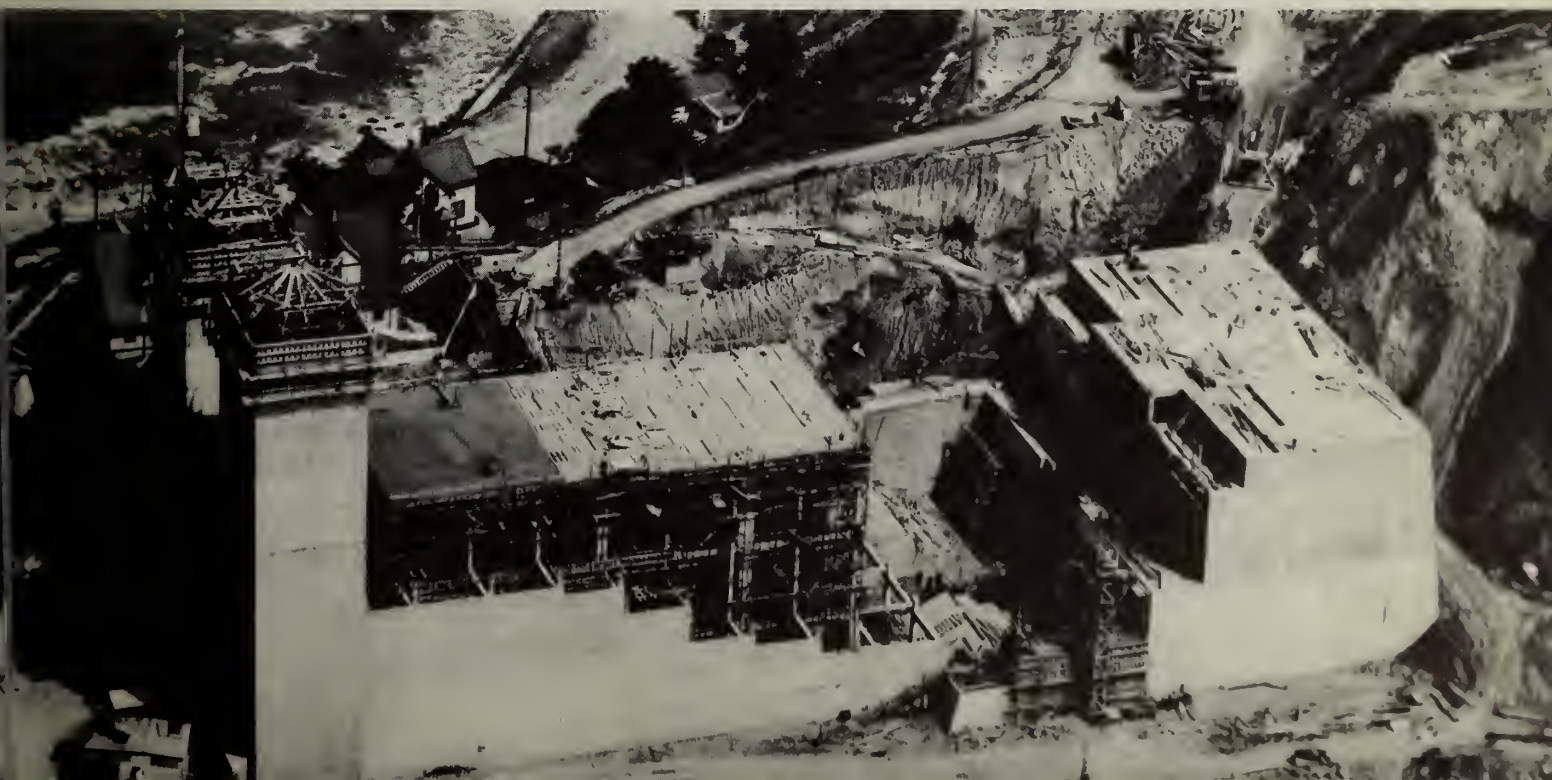
Construction of pylons at the shore-ends of the side spans was carried on simultaneously with the main anchorages. These pylons are a distinct structural feature of the bridge since they contain cable anchorages that restrain the cable vertically at equal distances from the main towers. This required anchorage steel to be set in the tops of these pylons and it was necessary to have this steel ready to receive the cables upon their completion.

The pylon on the San Francisco side is located partly inside and partly outside of the old granite seawall. The first step in its construction was to relocate and rebuild the old seawall to clear the pylon. The old granite blocks were reused in the new seawall. The seawall was constructed so as to be used as a cofferdam for the foundations of the pylon itself which consists of two hollow concrete shafts, each approximately 32 feet by 36 feet in cross-section, with walls in general two feet thick. These shafts were concreted in vertical lifts of ten feet each. A special movable wooden form was used that could be lifted, as a unit, to the next pour. The shafts were concreted to elevation 187 at which stage the anchorage steelwork constituting the cable tie-down, was erected with the pylon. Completion of the pylon could not be accomplished until after the cables were finished and the footwalks removed.

The pylon on the Marin side is located on the side of the hill in front of the Marin Anchorage. Footings for this pylon are founded on hard sandstone, well down and within the ledge of the cliff. Material encountered here was extremely hard and the footings were shafted down without shoring. This pylon also was constructed in two stages: the first stage being completed prior to cable erection and the second stage after the footwalks had been removed.

The Anchorage Housings are a part of the anchorage itself, and serve the useful purpose of protecting the cables below the splay points from the weather. Footings in

The San Francisco
Anchorage and
Anchorage Housing



all cases were built on firm rock at elevations that assured their stability in relation to the excavations on either side of them. The housings were constructed in two stages, the first stage included all construction that could be done without interfering with the operations of the cable contractor. The second stage was undertaken after the cables had been completed.

MARIN PIER CONSTRUCTION

The Marin Pier is located on a rocky shelf of basalt at the water's edge of a high cliff. To gain access to the site it was necessary to build a road 1700 feet long from the end of an existing road at Fort Baker. The construction of this road which was the first work done on the contract was commenced on December 22, 1932.

The plans called for the pier to be founded at elevation -20 . Since the site was partially under water, it was necessary for the contractor to first construct a water tight cofferdam around the site. On account of the rocky bottom and the exposure to the full force of the ocean, he elected first to place stone filled wood cribs in the deeper water to the outside of which a steel sheet pile cofferdam could be anchored. In the shallow water it was found possible to drive steel rails into the talus from which steel channel walers could be supported. The steel sheet piling was driven between these channel walers with whatever penetration of the piles could be secured. They could be driven deeply into the sand pockets between boulders but no penetration whatsoever could be realized where they encountered large boulders or the rock ledge.

Three wooden cribs were built at a shipyard and towed to the site. Upon a survey of the bottom by divers the cribs were altered as required to fit the contours, filled with large stone and sunk to position. With the cribs in place, the sheet pile cofferdam was completed by tying the walers to the cribs. This work was performed with a crawler type locomotive crane working on a wooden trestle built on top of the cribs.

As soon as the cofferdam was completed, it was unwatered and excavation started. The rock was blasted and the spoil was removed with a power shovel operating within the excavation. When the excavation had reached the elevation of -20 , it was discovered that rock was not exposed over a portion of the southwest corner of the foundation. Accordingly the overlying material was removed and the foundation bed stepped-off to meet the lowest elevation where solid rock was encountered at elevation -33.5 . The foundation steps below -20 were filled with concrete to that elevation and effectively keyed into the base slab above.

Because of the huge size of the pier it was impossible to concrete the base slab in one operation. Accordingly this slab which was 10 feet thick, was concreted in four sections. These sections were of such size that the concrete could be placed in horizontal layers approximately one foot thick and a complete layer deposited before the preceding one had reached its initial set. In this manner a monolithic mass of concrete was assured for each pour. All pour sections were keyed together by shear keys and steel dowels to insure unity of action.



On top of the base slab the pier shaft was concreted in two vertical lifts each consisting of four pours. The first, or bottom lift, was 20 feet high from elevation -10 to elevation 10 and the second lift was 34 feet high from elevation 10 to elevation 44. The pour blocks of each lift were so laid out that they overlapped the blocks in the pour below and thus with the aid of keys the entire mass was united. By dividing the pier into vertical blocks of comparatively small horizontal cross-section, a minimum of shrinkage cracks was assured. Also, by concreting alternate blocks, the intermediate key blocks filled the spaces without the usual voids caused by shrinkage.

The Marin Pier and
cofferdam. Lime Point
Lighthouse at the right

Embedded in the pier shaft above elevation -9 is the anchorage steel for the main tower columns. This anchorage steel consists of 107 tons of structural steel which extends above the top of the pier and is riveted to the main material of the tower. It was essential that it be held securely in position during the concreting operations and this was done with complete success. The various mats of reinforcing steel in the pier were supported by independent means which in no way affected the position of the main anchorage steelwork.

On account of the large, plane, exposed surfaces, form lumber with the rough side against the concrete was used. This gave the exterior a rough texture which does not reflect the light in such a manner as to accentuate minor irregularities in the surfaces of the work as would be the case with a smooth finish.

Since the top of the pier was required to be poured slightly above finished elevation to permit grinding down to a true plane, it was essential that the top surface be as near a true plane as possible in order that the grinding could be accomplished with uniformity. The upper two feet of concrete under the bearing plates of the tower were placed

after the main mass of the pier had been given time to shrink. A plane top surface with a maximum variation of $\frac{1}{4}$ inch was secured by setting wooden screeds not more than four feet apart over the bearing area.

The upper seven feet of the pier contains four heavy mats of reinforcing steel to take care of the horizontal force transmitted to the pier from the tower above.

Surface reinforcing was placed not less than 9 inches from the surface of the concrete so as to provide ample covering as protection against weathering.

SAN FRANCISCO PIER CONSTRUCTION

The site of the San Francisco Pier is 1100 feet off-shore at the narrowest point of the Golden Gate where the tide sweeps with a velocity of 6.5 knots and where there is no protection from the full force of heavy seas and ocean storms. The bottom of the strait at this point is bare rock—a ledge of serpentine from which all traces of overburden, except large boulders, have been swept away, lying 60 to 80 feet below low water; the pier footing was to be carried to 100 feet below low water. The site afforded scant facilities for construction equipment and operations. Thus, every proposed scheme for construction had to face not only the general problem of founding a pier in deep water on a rock ledge devoid of overburden, but also had to deal with strong tidal currents daily, and occasionally the battering of heavy seas incidental to ocean exposure.

It was required by the specifications that, "the rock surface shall be exposed to permit thorough inspection, exploration and excavation, and the depositing of the concrete in the dry. The pier shall be constructed by the pneumatic process." A minimum penetration of 20 feet into the rock ledge had been established as a requirement to insure ample confinement of the supporting rock. As to method of construction, analysis of the problem indicated the essential requirement to be sufficient protection of the pier site to permit excavating and leveling off a bearing surface 20 to 40 feet down in the rock, and provision for inspection of the foundation bedrock as an assurance of its security.

To afford the required protection the original design contemplated the construction of a concrete fender-wall around the pier site, founded on the properly benched-off, rock surface. This wall as designed had a thickness of 30 ft. up to elevation -45. Between -45 and -18 it was $27\frac{1}{2}$ ft. thick and above that the thickness varied from 14 ft. at -18 to 10 ft. at 15. This fender was to be 170 feet x 311 feet outside, with rounded ends, and 110 feet x 247 feet inside. Within this enclosure a pneumatic caisson 90 feet by 185 feet in overall plan dimensions with rounded and pointed ends, was to be sunk as a base for the concrete pier superstructure, the latter to begin at about elevation -50.

The original plan called for the construction of the fender wall in successive units by underwater methods and for pneumatic sinking of the pier base itself. As the work progressed, conditions made changes in the construction procedure necessary, and from

time to time the original scheme was revised and developed. The final construction retained the method of constructing the fender wall about as originally specified, but discarded the pneumatic method for constructing the pier base.

One important result of these changes was to eliminate the pier base as a separate unit and, in effect, to integrate the fender with the deep tremie concrete mass, poured within the fender as supporting base for the pier.

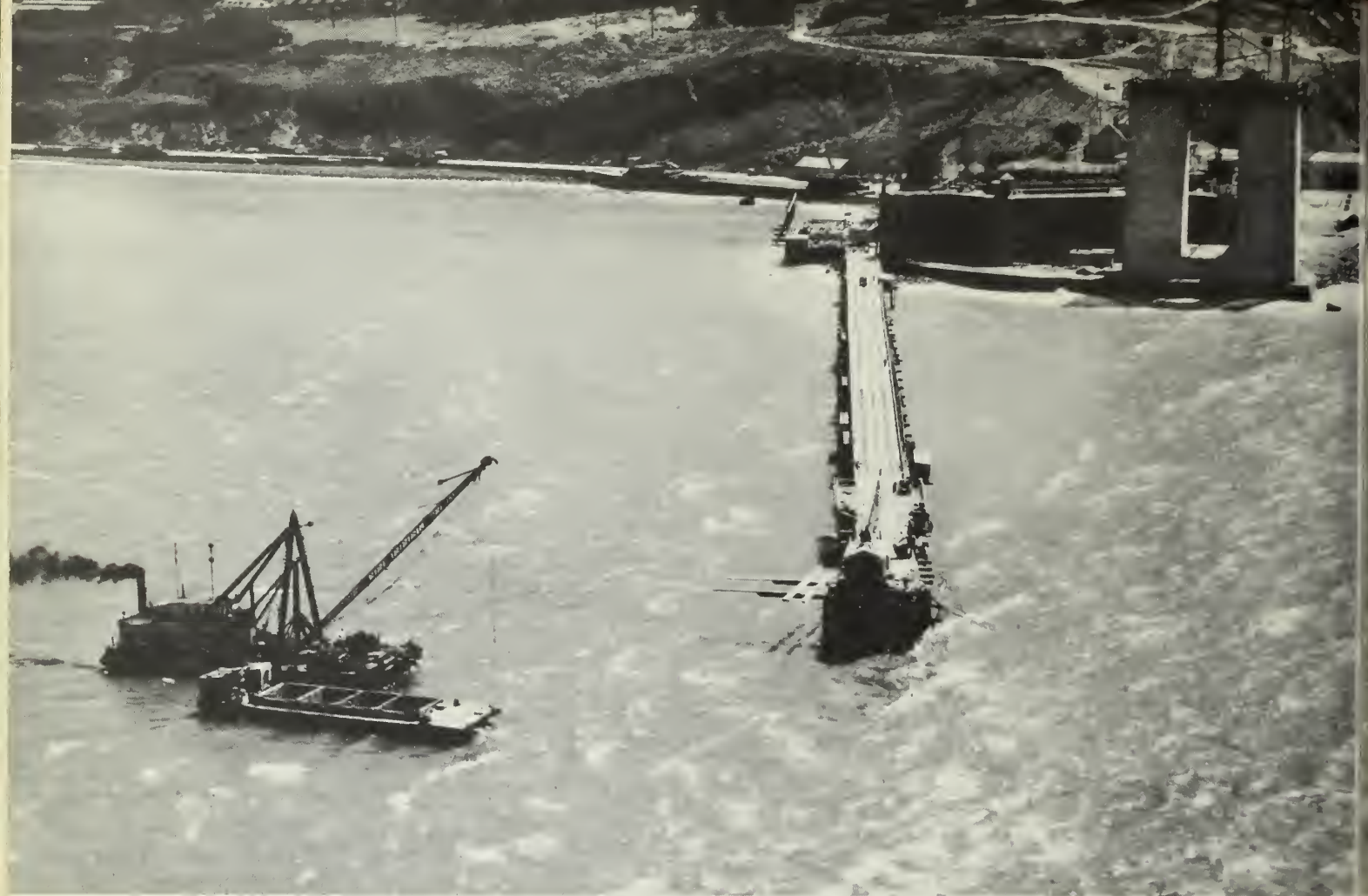
ACCESS TRESTLE: The use of floating equipment at the pier site for anything but dredging was precluded by rough water. The first step in construction was to provide access to the pier site by a trestle which it was planned to continue around the pier site. A steel trestle was therefore built out from shore to the pier site; the deck of this first trestle was at elevation 15. The original plan for trestle-bent footings was to drill holes in the rock slightly larger than the steel columns and, after insertion of the columns to place concrete around them, but this plan was abandoned when no drills could be found that could clear the holes effectively as they were drilled.

A submarine bombing scheme then was developed, which consisted of dropping a small bomb on the rock at the pile location, driving it into the rock with a heavy follower, then firing it, and shattering the material with successive blasts of this sort until it was possible to drive a steel tubular-pile footing into the shattered rock. This method was successful. The steel tubes were driven into position, and the bottoms of the columns were inserted in them. After each bent was erected, the tubes were filled with concrete. Under this plan, trestle erection proceeded rapidly and was completed October 19, 1933.

Just as the trestle was completed, a 2000 ton steamer ran through it about 400 feet offshore. The collision carried away six spans entirely and because the trestle deck had been securely fastened together, the impact of collision thrust the remaining portion of the trestle about 6 feet inshore. Immediately after this accident, repair of the trestle was begun. Wreckage of the demolished bents was removed by divers. The destroyed section was replaced by timber bents to save the time required for the fabrication of steel bents.

The outshore bent of the access trestle was built in the form of a steel tower that weighed about 50 tons and was designed to serve as a guide for lowering into place forms for the first unit of the concrete fender ring.

The outer vertical face of the guide-tower was provided with two 85 pound rails down which the forms for the first section of the fender were to be lowered; forms for succeeding sections could then be lowered down similar rails on the leading face of the previously concreted section. The tower was supported in the same manner as the trestle bents. Four footing pipes were first driven into the rock in holes made by small pilot bombs; then the tower was lowered and the legs were guided into the pipes by divers. Difficulty and hazard attended the placing of this 50 ton, 115 foot high, steel tower, because of the force of the waves which swept in from the open sea, and the tides. Entering the tower legs into the foundation pipes required great skill on the part of the



Commencement of
construction of
fender-ring at end of
access trestle

four divers doing the work, as well as close coordination between these divers and the crew above water. Working periods were limited to slack water between ebb and flow of the tide. The tower was placed on the sloping south side of the pier excavation mentioned later; the north or lower legs were founded at elevation -90 (about 10 feet above the bottom of the pier excavation) and the upper or south legs at elevation -70 on the adjacent sloping surface.

EXCAVATION METHODS: Excavation of the pier site was started with a floating dredge coincident with construction of the access trestle. Measured at the longitudinal center line of the bridge, the water depth at the south or land side of the pier was 50 feet and at the north side 80 feet. The rock ledge was of such hardness that it could not be removed without blasting. A method of underwater blasting was developed which involved blasting with small pilot bombs until a sufficient depth of rock had been loosened so that a large bomb containing some 200 pounds of high explosive could be driven into the hole well below the adjoining rock surface. A novel phase of this method was the driving of the large bomb into place by means of a 2500 pound hammer, after which the wire leads to the blasting cap in the bomb were recovered by a diver and carried to an electric blasting machine on the surface.

After blasting, the loose rock was excavated by a 5 cubic yard bucket with specially reinforced manganese-steel teeth and lips. The great amount of wear on this bucket

attested the hardness of the rock. In general, a depth of 15 feet could be excavated after one layer of bombs had been fired 20 feet below the rock surface. Successive layers of bombs were staggered so as to remove projecting high points. This program of excavation, carried down to elevation -100, included the area on which the guide-tower was to be erected.

THE FENDER RING: Several methods for constructing the fender previously described as a means for protecting the pier during construction, were developed at different stages of the work. The original plan was to erect the wall, 30 feet thick at the bottom, entirely around the pier site and within this fender to build the pier base on a pneumatic caisson. The fender wall was to have been founded on the original surface of the rock, and when the enclosure was completed, a pneumatic caisson was to have been built in place and sunk through a sand leveling fill placed inside the fender. The caisson was to have penetrated 20 feet or more into the rock ledge, thus placing the pier bottom at elevation -100.

The specification contemplated construction of the fender in 22 separate units each about 30 feet square in plan. The first revision (See diagram "First Revision" page 143), under which the work was started, provided for an extension of the inside face of the fender to the bottom of the pier excavation, adopted the open dredging method of excavation and left only the final preparation of the pier foundation to be done under air pressure. The concrete was to be placed in one continuous pour for a height of about 80 feet in each of the 22 separate units which were to comprise the fender ring, ultimately extending from elevation -100 to elevation 15, a total height of 115 feet. The forms for these units were to consist of a series of steel boxes each 20 feet high, open top and bottom, fitted with jaw castings on one side to engage the guide rails on the face of the guide tower. The bottom of the first box form was carefully shaped to make a close fit with the bottom of the bay, as determined by very accurate soundings.

To insure a close fit between successive sections, it was planned to lower the first box until the top was just awash and then to erect a second 20 foot section on top of the first. The two sections were to be fastened together by riveted splices of the columns in the corners. The assembled pair of units, each with jaw castings engaging the rails, was to be lowered until a third and subsequently a fourth section could be added, thus making a tier of sections fastened together. When the tier rested on the bottom the top-most edge could have been at elevation -20.

This program was followed up to the point where four sections (80 feet of continuous forms) were suspended in engagement with the rails of the guide-tower. The plan anticipated that when this combined form had been rested on the bottom it would be filled with concrete by the tremie method up to elevation -20 without interruption. After this first unit, 80 feet high had been completed, the same procedure of lowering successive boxes and riveting them together was to be followed for succeeding units until all 22 were in place, thus completing the fender. The point of note in this

plan is that each individual unit was to have been concreted continuously for a height of 80 feet before any concrete was placed in the next unit. Difficulties met in this plan will become apparent in the following.

After the legs of the guide-tower had been entered into the pipes embedded in the rock foundation, they were concreted firmly in place. The steel form for the first fender unit, weighing about 40 tons, was successfully placed in the guide rails and lowered until its top was awash. Additional sections were added, and three had been riveted together as planned before the elements interfered. Then a storm that rolled tremendous waves through the Golden Gate caught the guide-tower supporting this eccentric load of three forms. The tall tower was buffeted to such an extent that it began to oscillate about 6 feet backward and forward. The oscillation evidently caused the displacement of the foundation pipes in which footings were supported, and on October 31, 1933 the entire tower and the forms attached to it were carried away.

The contractor at once proceeded to recover the guide-tower and forms. No sooner had they been salvaged than another storm caused a repetition of the tremendous rollers sweeping in through the Gate, and this time all but 600 feet of the access trestle was carried away. Coincident with the removal of the wreckage, the contractors' engineers began to develop plans for a new and stronger access trestle.

In reconstructing the trestle, it was decided to use timber not only because of the saving in time by avoiding the delay necessary for fabrication of new steel bents, but also because the round timber piles offered less resistance to the strong tidal currents than had the rolled-steel "I" sections used previously. Further precautions included raising the trestle deck height by 5 feet so as to keep the stringers well above the tops of high waves, and guying the bents with steel cables in both directions. The reconstructed trestle deck was 25 feet above mean lower low water. The new trestle was started December 15, 1933 and was completed to the pier site March 8, 1934. The last three bents of the trestle adjacent to the guide-tower were made of steel and were very rigidly braced both above and below water.

Failure of the guide-tower had demonstrated the difficulties of supporting the fender on the sloping sides of the pier excavation and suggested the wisdom of having fender footings on a level surface at the same elevation as the pier bottom.

NEW METHOD OF CONCRETING FENDER UNITS: Accordingly, a modification of the fender design was developed, and the plan of construction was changed to meet it. Departing from the original plan for a continuous pour in each vertical unit, it now was decided to lay up the fender wall in courses about 20 feet high. The courses were to consist of blocks 30 feet long corresponding in plan to the original 22 fender units. The start of the first course was to be made at the point where the first vertical unit had been started and it was decided to include in this footing the columns of the guide-tower. The base section of the first fender unit, therefore, was formed and concreted in two parts, an inshore and an outshore section.

As part of this plan the area excavated to elevation -100 was increased to permit the entire fender ring to be supported on a level foundation at the same elevation as the bottom of the pier, instead of only the inner portion of the ring as was done at first. (See diagram "Second Revision" page 143.) Also, under the modified plan, the fender units were to be built up to elevation -60 around the entire elliptical ring before any appreciable amount of exposed wall was raised above that level. This is the plan that was finally carried to completion.

It was proposed that, after the fender ring had been closed at elevation -40 the westerly end would be built up to its full height of elevation 15 and then the steel framing over eight units at the east end above elevation -60 would be temporarily removed to permit entry of the caisson, which would have been built at a shipyard, into the fender ring. After the caisson had been floated into place, the plan was to replace the steel framing on the east end, complete the concrete fender ring and then load and sink the caisson within the fender enclosure.

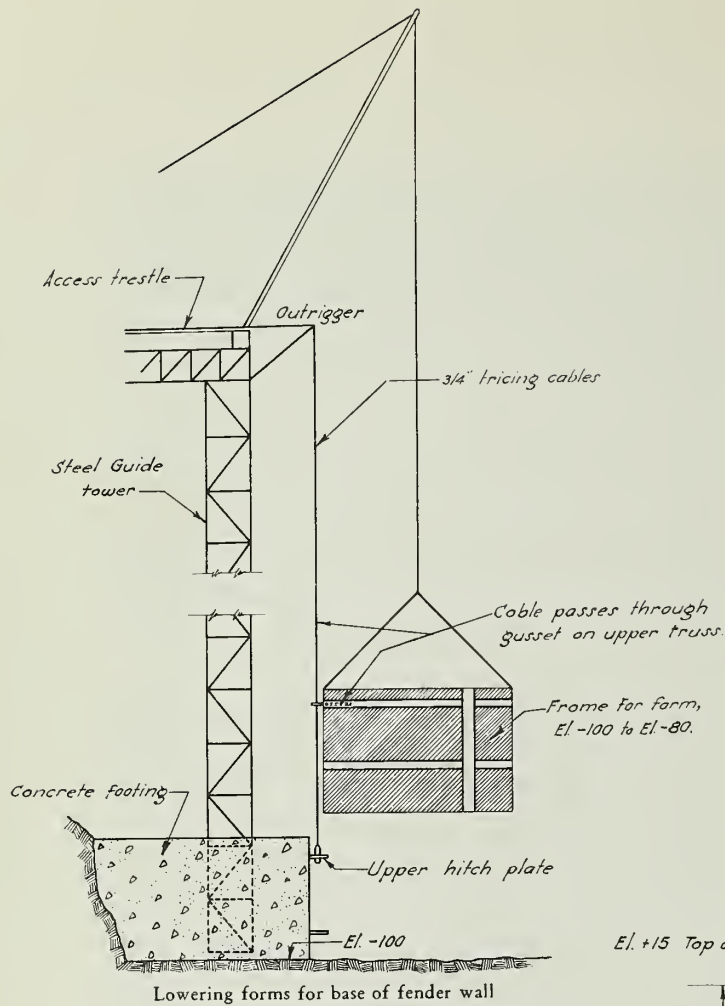
It was found from careful study of the stability of the fender sections under all weather conditions that, because of their greater height resulting from dredging the full fender area to elevation -100 , an increase in the fender base-width would be necessary. In the "Second Revision," therefore, the base section of the fender ring was increased in thickness by a 10 foot offset inward and by whatever outward offset would be necessary to carry the concrete up against the wall of the excavation within which the fender was built.

The inward offset of the fender base, with its top at elevation -80 , extended into the area that was to be covered by the caisson. In consequence, the caisson could no longer be sunk down to the rock surface but would have to come to rest on the ledge of the fender base. It was contemplated that the space between the caisson and the inner face of the fender would be filled with concrete from elevation -80 to -50 by tremie, to seal the joint in order to make possible to hold the compressed air while gaining the foundation. Concrete piers were planned to be built within the fender area under the intersections of the caisson cross-walls, to assist in carrying the weight of the caisson until the working chamber could be sealed.

The excavation over the fender base area was to be carried to elevation -100 in general, but to -105 at the east end of the fender and to -110 at the northeast sector where the original rock surface dipped down in a narrow depression to about -95 . Grouted riprap was to be placed between the fender and the side of the excavation.

The revised plan for placing concrete in the fender ring involved building a considerably larger surface of formwork under water. Because most of the steel boxes planned for use in the original concreting scheme had been fabricated, the contractor prepared to utilize as many of them as possible. Accordingly, these boxes were used between elevation -80 and elevation -40 .

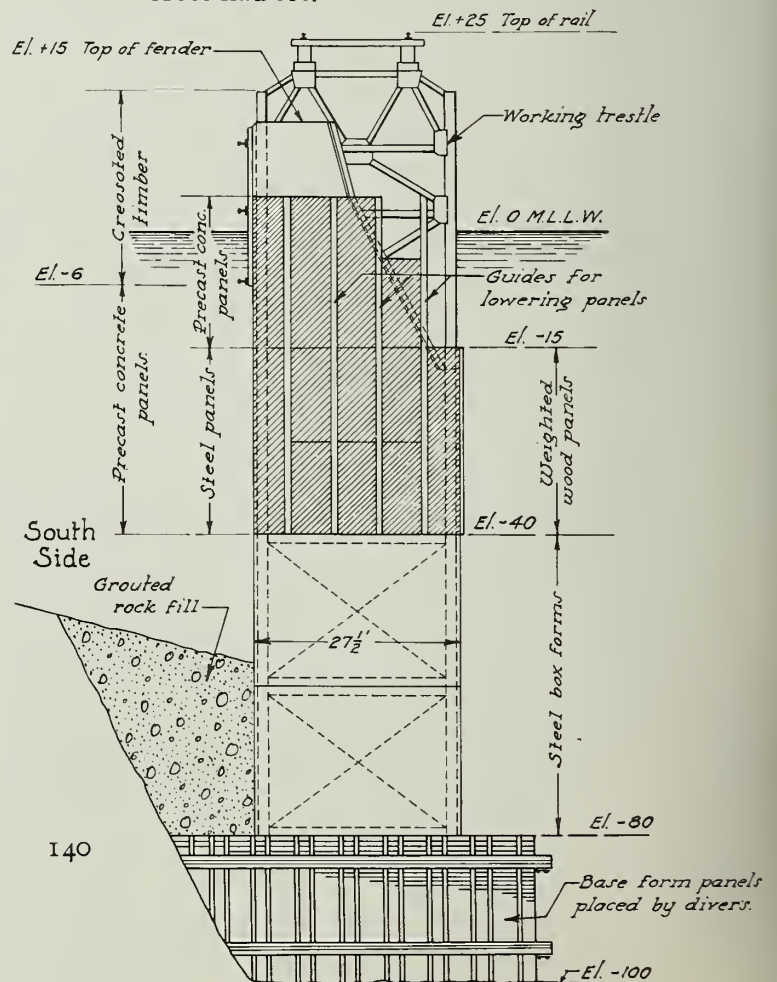
The larger cross-section of footings for units under the revised plan called for the development of a lighter form. Therefore, for that portion of the fender below elevation



The bottom form for the pioneer unit was placed in two sections. The first of these encompassed the legs of the guide tower; later the first steel box was lowered and rested upon the bottom form, after which concreting of the remainder of the base of the first unit and the steel box was made as a continuous pour. On all the remaining sections the first, or bottom steel box, was concreted integrally with the base block.

The second steel box, constituting the form for the fender wall between elevation -60 and elevation -40 was concreted integrally with an adjoining box on the next unit so that the second course of the wall was built up

—80, each form was built as a relatively light, wood and steel frame, lowered into position on tricing lines, two to each form, which served to guide the forms to their proper positions. The frame alone offered a minimum of resistance to tidal currents as it was lowered. Once the frame was in place on the bottom and fastened to the adjoining units, the sides were built up with wooden panels about 5 feet wide. These panels were weighted with concrete blocks to overcome buoyancy; and were placed by divers using a fastening adapted to quick manipulation under water. All of the wooden panels and the principal members of the steel frame were stripped and re-used after the concrete had set.



Typical section of fender and types of concrete forms used



Above: Building fender—working out in both directions from initial unit. Below: Pouring tremie concrete in fender. Concrete poured into hopper just above the water is deposited in the forms 100 feet below



of blocks $27\frac{1}{2}$ feet x 60 feet in plan and 20 feet high.

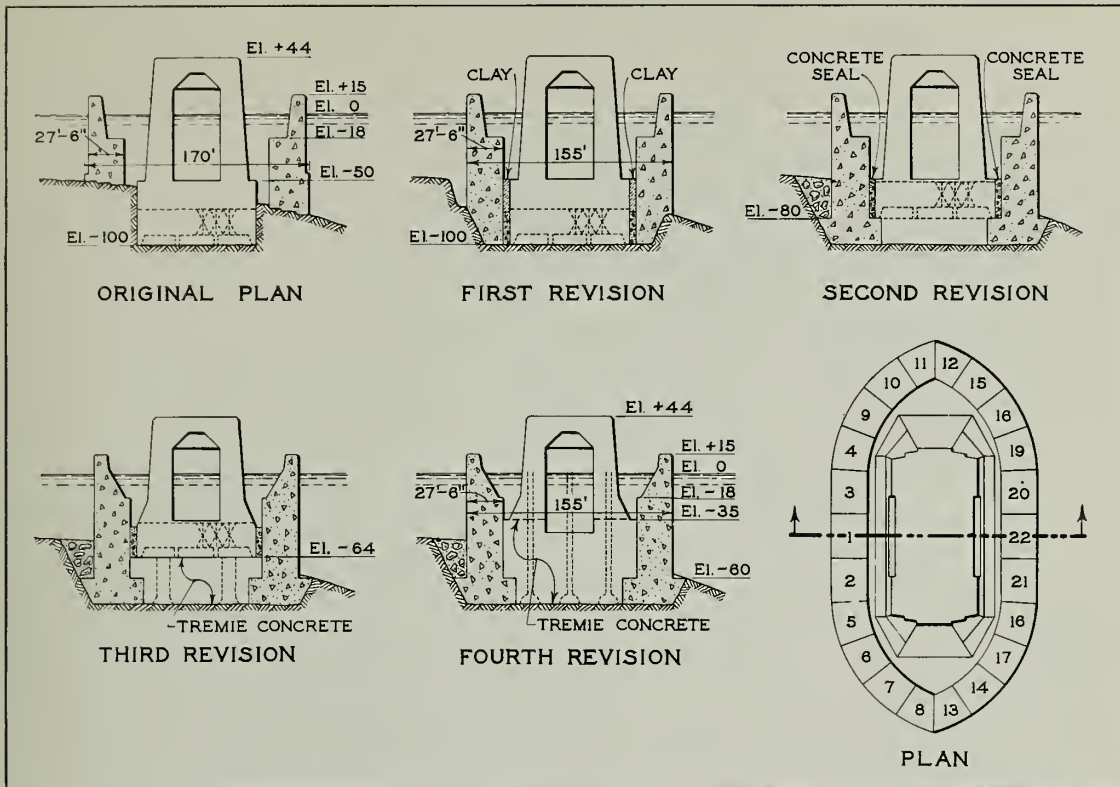
Above elevation -40, an entirely different plan was used and the wall was built of blocks approximately $27\frac{1}{2}$ feet x 60 feet in plan and 25 feet high. The bents and bracing in the trestle over the fender side were so designed that they later served as guides for form panels. This skeleton steel frame offered a minimum of resistance to the current during construction of the ring below elevation -60 and was thoroughly braced so that in building that part of the fender above elevation -40, two 20 foot sections of the panel forms could be erected simultaneously with safety. The forms were designed to withstand a horizontal pressure of 130 pounds per square foot. Above elevation -40, form panels of three kinds were used: steel, weighed timber and precast concrete. The latter were made 6 inches thick and of widths to fit the spaces between the vertical guides. The precast concrete panels were used principally in forming the outside of the fender where a smooth concrete face was required.

The entire fender wall was firmly bonded together with keys between adjoining units and by staggering the vertical joints. In addition, the top portion of the wall above elevation -2 was heavily reinforced with steel bars. This reinforcement ties the tops of the units together and horizontal arch action will be developed in case the structure is rammed by a ship. Above elevation -40, enough vertical reinforcing steel was used so that each unit would stand securely as a vertical cantilever in resisting both wave and current forces.

TREMIE FENDER SEAL: After the first concrete was poured in the pioneer unit on March 31, 1934, progress in the fender ring proceeded according to schedule, to the point where the eastern end of the ring was concreted to elevation -40 and the remainder of the fender wall had been brought up to elevation 15, the ultimate height. This was in accord with the earlier plan to leave the eastern end open, to permit the floating in of the caisson as the contractor had elected to do.

An important change was made after the final plan of fender construction had been adopted. It was decided to abandon the purpose of exposing the entire area of foundation bedrock within the caisson, and instead to place a tremie concrete seal within the fender up to elevation -64. The base of the caisson was to be set on the top of this seal concrete instead of on the fender base ledge at -80 as previously intended. However, in order that the foundation rock could be inspected (and, if necessary, further excavation done), a series of eight inspection wells, 4 feet in diameter was to be provided in the concrete seal, each ending in a 15-foot steel hemisphere resting on the rock surface. These shafts could then be entered from the caisson working-chamber under the protection of compressed air and the foundation rock examined. This decision was reached in June, 1934 (See diagram "Third Revision").

Results of the dredging operations had indicated that the material under the entire pier area was uniform. Hence it was felt that the area made accessible by the eight inspection wells would give ample characteristic evidence of the nature of the founda-



San Francisco Pier and Fender. Diagram shows original plan and subsequent progressive development during construction

The fender wall has been completed and the tremie seal is being poured



tion rock. In case inspection indicated any necessity for going deeper with the pier foundation it still would be possible, by excavating in the inspection chambers under air pressure and using the seal concrete as the deck of the working area, to carry the pier footings to a lower level.

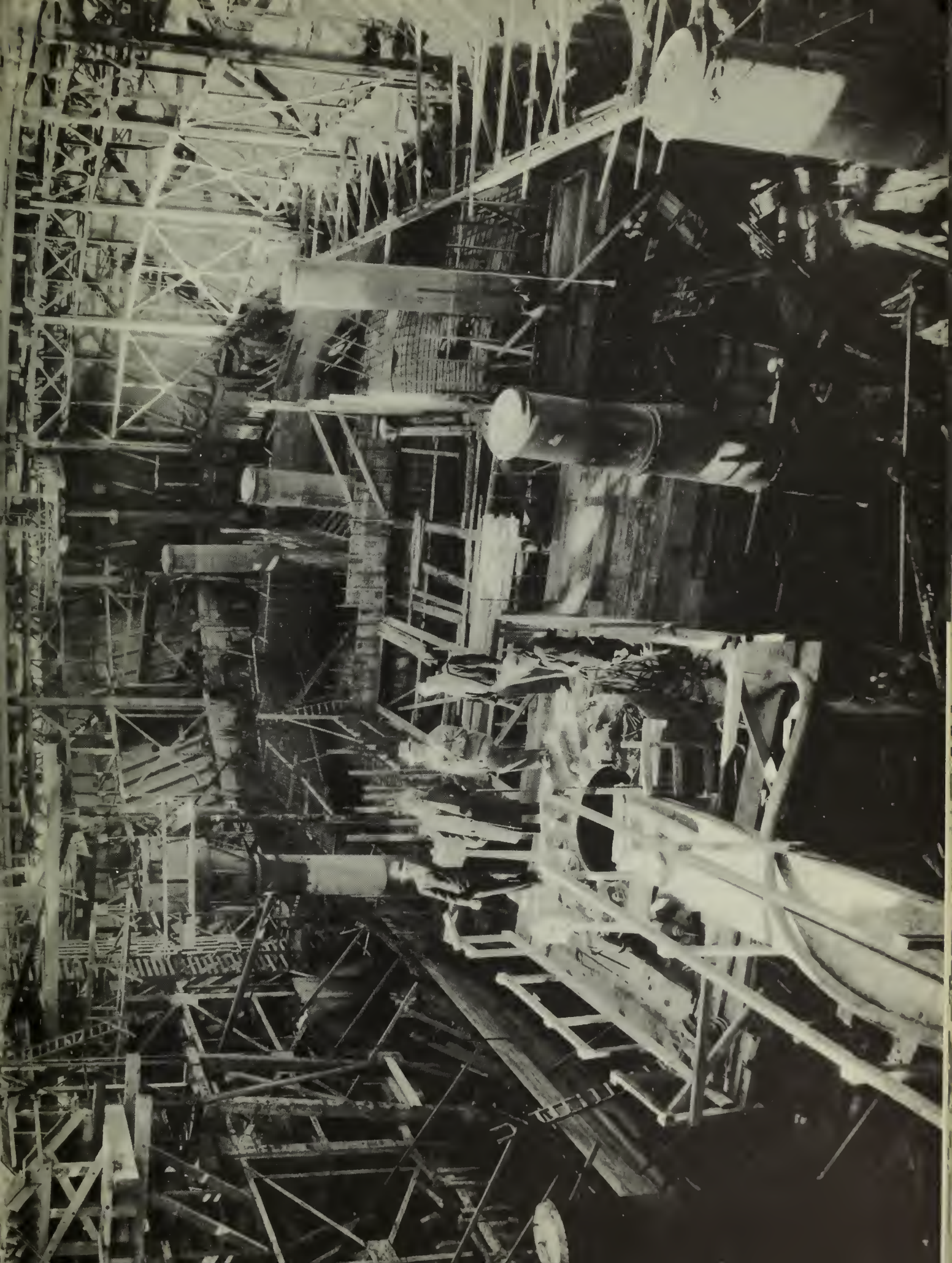
When the entire fender ring had been brought up to elevation -40, and a few units at the west end had been continued to elevation 15, the effect of currents and wave action on the bottom was so greatly diminished that divers were able to prepare the surface of the bottom at the pier site for the tremie seal. The area to be covered was too great to attempt a single continuous pour and therefore it was divided into seven sections, each 30 feet x 90 feet in plan. Wooden forms, similar to those used in the construction of the fender bases were designed for three of these sections and were placed by divers. Details of these forms were such as to permit the removal of all wood used as forms in the tremie seal. The first tremie sections were carried up to elevation -64.

CAISSON MOORED IN ENCLOSURE—THEN REMOVED: On October 8, 1934, with all skeleton steel bents removed from the east end of the fender, a caisson was towed to the site and moored within the fender enclosure. Preparations immediately were begun to close the east end of the fender. Before this could be done however, a violent storm arose and the surging of the caisson within the fender enclosure caused by wave action made it impossible to hold it firmly in place with mooring lines. It battered the steelwork and concrete of the fender with such force as to threaten to wreck itself within the fender enclosure. Several of the fender frame members were badly bent, and the inner face of the concrete gouged deeply.

Because wrecking of the caisson within the fender at this time would endanger the rest of the work and seriously delay completion, the contractor secured permission to remove the caisson at once and abandon that part of the construction-plan that involved its use. Accordingly, the caisson was removed and the closure of the east end of the fender wall was begun immediately.

The fender ring was completed on October 28 and by November 4 the tremie blanket had been raised to elevation -64. In lieu of using the caisson, the contractor planned to pump out the fender enclosure, using the fender wall as a cofferdam. To do this, it was necessary to add more concrete to the seal, raising its upper surfaces to elevation -34. This additional concrete was put in in seven units, overlapping construction joints in lower portions of the seal and increasing the thickness over the shelf of the fender base. The several changes made in the seal and the shape of the pier shaft above elevation -35 served to distribute the load over the base of the entire fender in addition to the base of the pier. The original bearing area of the pier, 16,000 square feet was thus increased to 38,000 square feet. (See diagram "Fourth Revision").

FENDER UNWATERED AND PIER COMPLETED: On November 27, 1934, the pier area within the fender wall was unwatered; the wall was found to be remarkably



tight, and very little pumping was needed to keep the bottom dry. The shafts of the eight inspection wells were fitted with air locks to permit of access to the rock surface under air pressure.

Examination of the foundation within these wells then was made by the engineers and by Dr. Andrew C. Lawson, consulting geologist. In addition, pressure tests were made by hydraulic jacks braced against the roof formed by the hemispherical chambers. It was concluded that the foundation rock was adequate in all respects and accordingly the eight inspection chambers and shafts were filled with concrete.

Within the open cofferdam formed by the fender ring and the concrete seal, forms for the pier proper were built. On January 3, 1935, the pier had been concreted to its final height, elevation 44. Quantities involved in the south pier construction were as follows:

Concrete in pier and fender, cubic yards.....	130,000
Reinforcing steel, tons.....	647.9
Rock excavation for pier and fender, cubic yards.....	45,000

All underwater concrete was deposited through tremies. In general, each position of the tremie was expected to deliver to an area of 900 square feet. A concreting rate of 100 cubic yards per hour was common. To test the uniformity of tremie concrete at various distances from the tremie pipe cores were taken with a diamond drill. These cores showed equal distribution of coarse aggregate as far as 35 feet from the tremie pipe and showed a compression strength of more than 4000 pounds per square inch in 28 days.

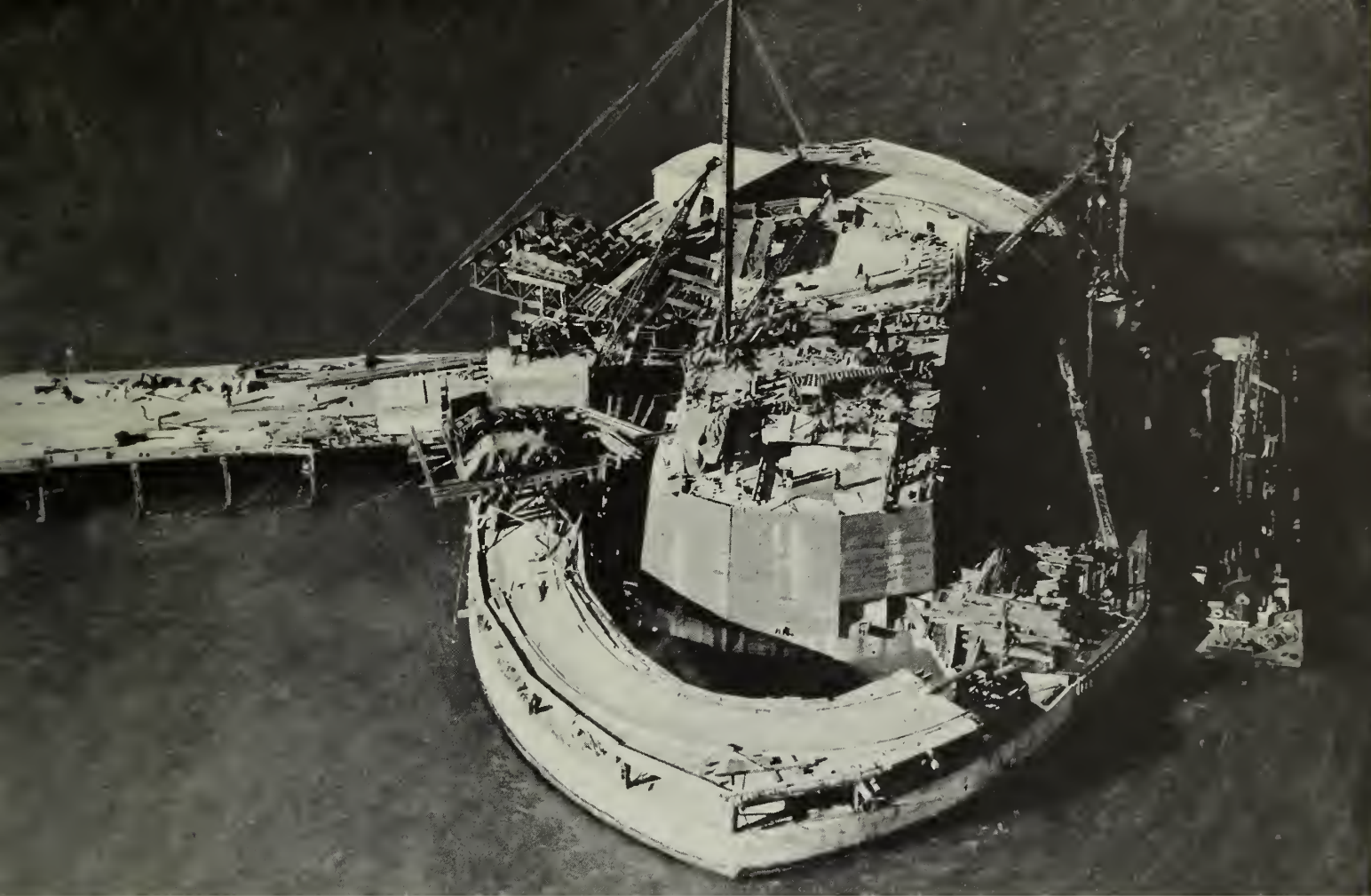
Concrete was batched at the plant of the Pacific Coast Aggregates Co. near the bridge site and was delivered by trucks that mixed in transit. All concrete placed in fender and pier contained 7 sacks of cement ($1\frac{3}{4}$ barrels) per cubic yard.

High-silica cement was selected, and the results justified this choice because there was decided advantage in the workability of the concrete where large tremie pours were made. Tests indicated that the cement would show a high degree of resistance to sea water.

TOWER ERECTION

Fabricated steel was shipped from the contractor's plants at Pottstown, Pa., and Steelton, Pa., by rail to Philadelphia and thence by water via the Panama Canal to Alameda. There it was unloaded and stored until required at the site. Steel was delivered to the erector in barge loads of about 500 tons each, which was the limit of storage at the tower pier. An 85 ton stiffleg derrick, with a 100 foot boom, unloaded the barges and rehandled the steel as necessary to serve the erection traveler.

Preparation of the pier top to receive the tower base slabs was a part of the work included in the tower contract. The specifications required the top surface of the concrete under the tower base to be ground to a level plane. This was done with a carborundum grinding wheel mounted on a trolley. The trolley supporting bridge itself



was arranged to move on rails accurately leveled. With this equipment the bearing area under each tower shaft, amounting to about 1200 square feet, was finished to the required level plane with variations of less than one-thirty-second of an inch. As soon as the tops of the pier under the bearing plates had been surfaced, the base plates were set in a thick coating of freshly applied red lead paste. By varying the thickness of this red lead cushion, the plates were set to an accuracy of .003 inch. After the plates had been set, the steel dowels, 4 feet 6 inches long and 6½ inches in diameter, were set in holes provided in the top of the pier and grouted in position.

The erection traveler comprised two steel trusses 79 feet long and 33 feet deep extending transversely to the bridge. They were spaced 28 feet apart and joined together with suitable cross bracing and struts. This traveler supported two stiffleg derricks, each of 85 ton capacity equipped with 90-foot booms. Provision was made in the tower shafts to support the travelers at predetermined elevations about 35 feet apart. Preparatory to raising the traveler from one position to the next, two beams were placed across the uppermost tower sections, one above each end of the traveler. Each end of these beams was equipped with four sheaves and these with four corresponding sheaves at the lower four corners of the traveler, were rove with one inch wire rope leading to the hoisting engines on top of the pier.

The derricks were independently operated by two electric motor-driven four-drum hoists, located at the base of the tower. Each derrick had its own raising gang and electric

The San Francisco Pier completed and preparations being made for erection of the San Francisco Tower

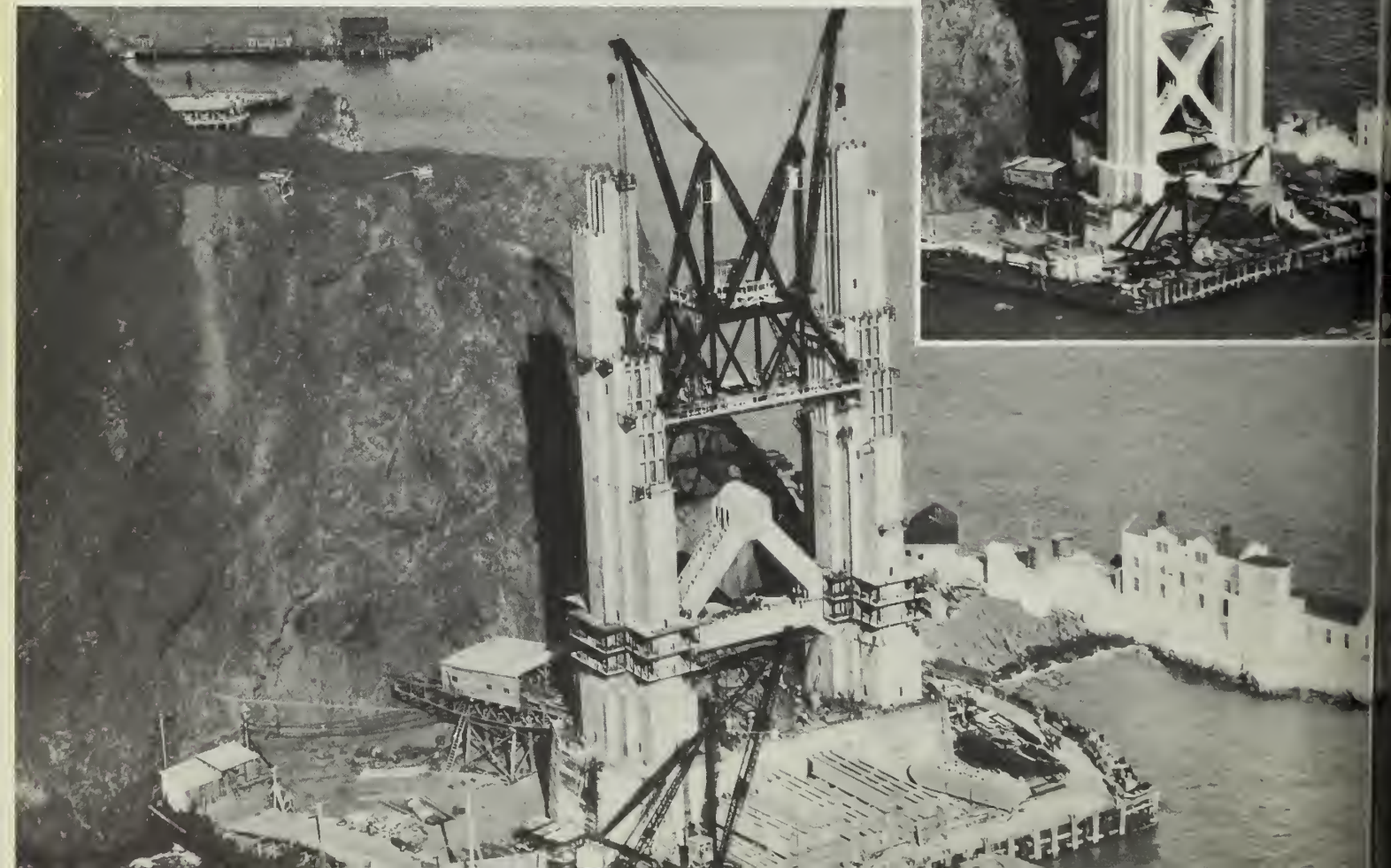
signaling system for transmitting hoisting signals from the derrick to the engineer below.

As soon as the steel erection had reached a height of 200 feet, the connections were made to the anchorage angles projecting from the top of the pier. These connections involved the pre-stressing of the angles in tension to a maximum of 105,000 lbs. per angle, then drilling holes from the solid in the tower webs, after which the connections were riveted.

It was found that almost perfect bearing between the milled surfaces could be secured by proper manipulation. Often this consisted merely in striking the tops of the sections with a pile driver. In some cases heated structural hitches were applied to the two sections so that they could be drawn together as the hitches cooled. Riveting was permitted when the opening of the sections was reduced to .006 inch or less, but it was found that after riveting had started the openings were closed under the vibration and heat of the riveting process.

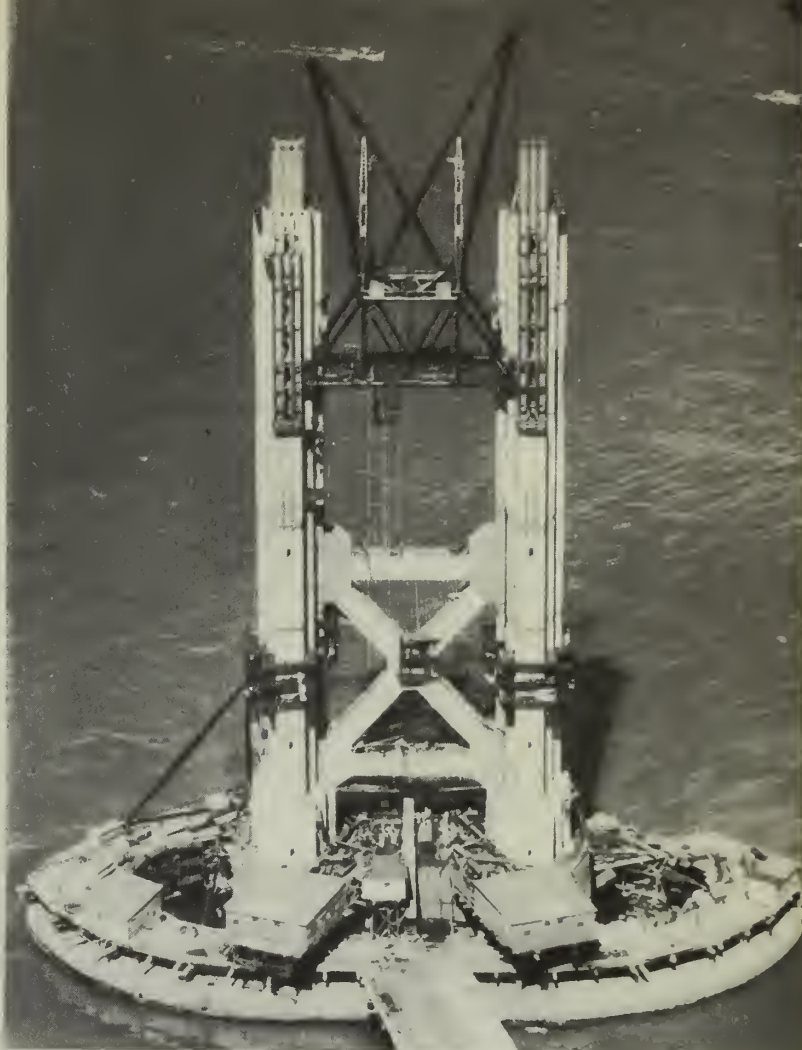
Erecting
the Marin
Tower

Rivets in vertical seams were not driven until all horizontal joints below had been riveted. There were



about 600,000 field driven rivets in each tower. The rivets were heated in coal-burning forges located on scaffolds outside the tower and passed to crews within the tower by pneumatic rivet-passers through holes $6\frac{1}{2}$ inches in diameter and left in the tower webs for that purpose.

Special provisions were made for the safety of the men during the erection. Every man was required to wear a hard hat to minimize head injuries from falling objects. Because of the confined space within cells, extra precautions were taken against lead poisoning. This ailment was suspected in several workmen and as soon as its likelihood was discovered, all men were examined physically every two weeks and blood counts taken. All riveters were required to wear respirators, and provisions made so that hands could be kept clean to prevent hand to mouth infection. As a result of this experience on the Marin Tower, the paint



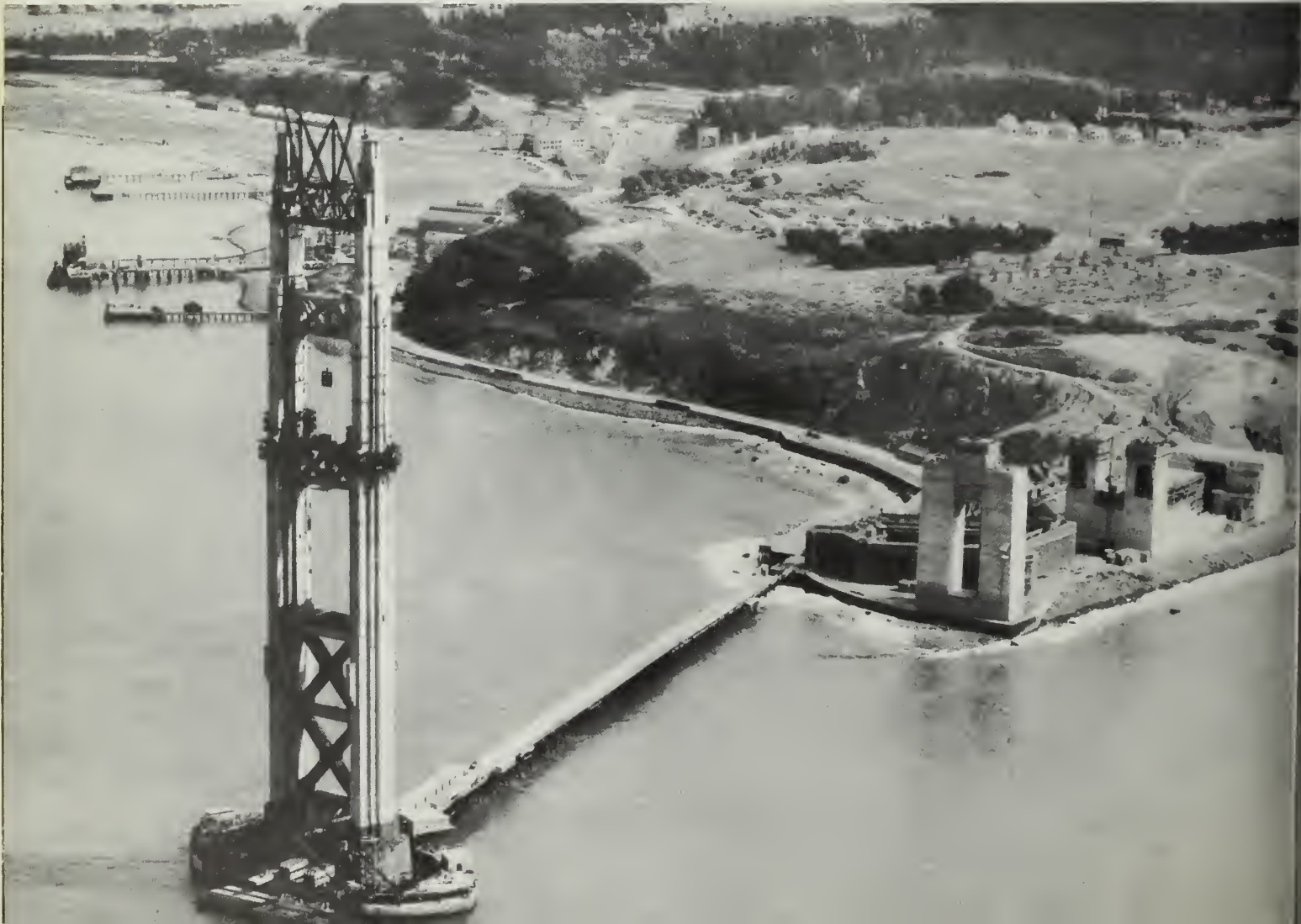
on the splices of the San Francisco Tower, then in the process of fabrication, was changed from red lead to iron oxide. It is believed that the erection of the two towers, involving 44,000 tons of steel without a fatal accident, established a noteworthy record.

In accord with the program and for reasons previously explained, the Marin Tower was erected first and by the placing of the saddles on its top in November, 1934, was then completed in readiness to receive the cables.

The completion of the San Francisco Pier on January 8, 1935, found the steel erector ready to begin the erection of the San Francisco Tower. A working platform and unloading derrick had been erected on the outward side of the fender. Grinding of the bearing surfaces was rushed by working double shift and every possible expedient was used to accelerate the construction of this unit of the work. A bonus agreement had been entered into between the District and the contractor, since the early completion of the San Francisco Tower would hasten the date of opening of the bridge.

The same traveler and other equipment used on the Marin Tower were used in the erection of the San Francisco Tower. The tower was erected ready for the cable contractor on June 28, 1935.

While the San Francisco Anchorage and Tower were progressing, work was also being done on the Presidio Approach Road. A portion of the Low Viaduct can be seen in the left-background and the High Viaduct piers can be seen in the center-background



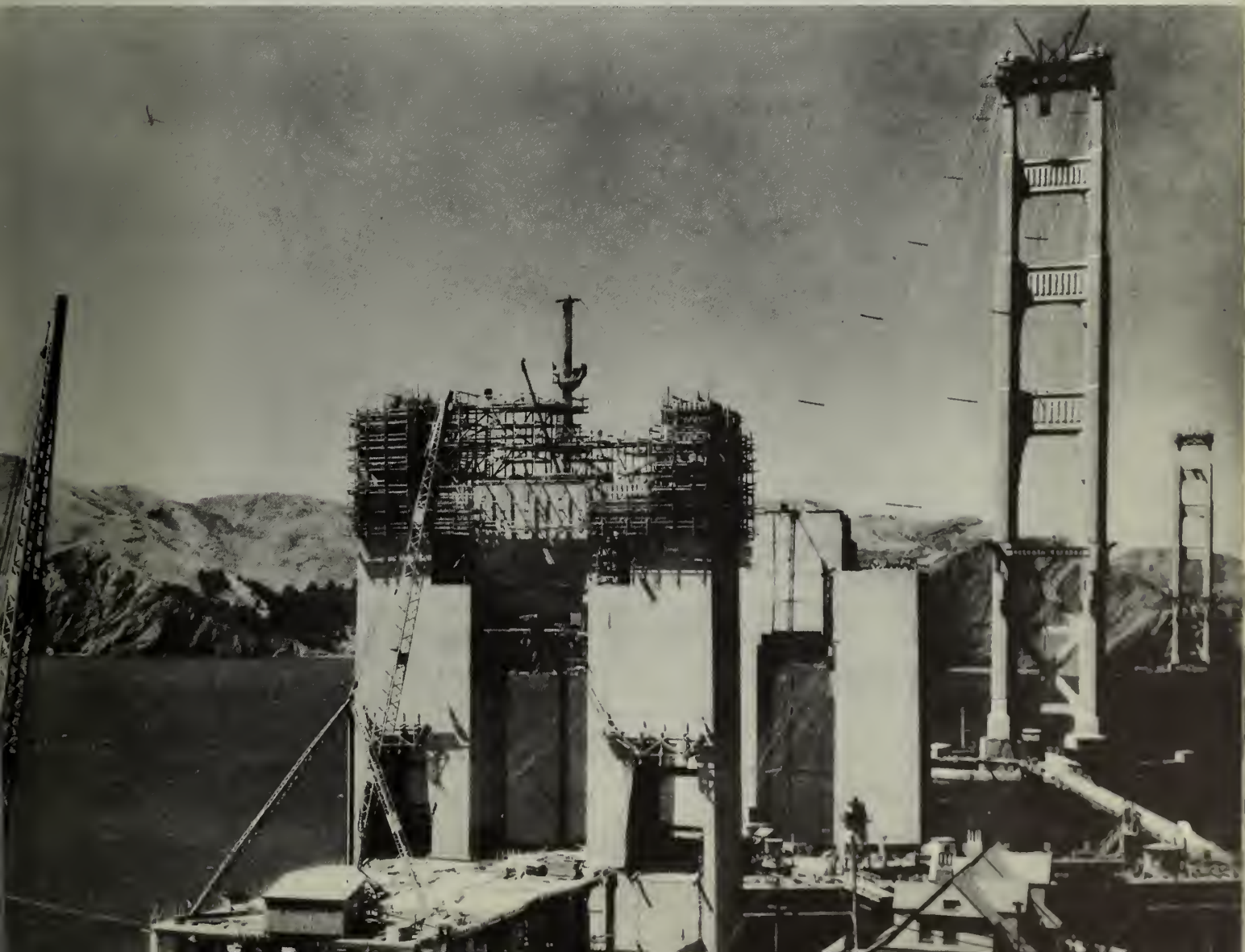
CABLE CONSTRUCTION

The cables are the largest built to date. Each of the two cables is thirty-six and three-eighths inches in diameter (over wrapping at a distance from the cable bands) and 7650 feet in length between the centers of the eye-bar pins at opposite anchorages. Each cable is made up of 27,572 parallel wires distributed into 61 strands. The great lengths of spans, combined with certain unique weather conditions encountered at the site, introduced new experiences in cable construction.

At the ends of the side spans, the cables are maintained at a constant elevation by means of tie-downs instead of bent or pier supports. The cables terminate in anchorages of the gravity type, each strand being held at the end by a cast steel strand shoe pinned between a pair of heat-treated eye-bars.

The unloaded position of the cable required that the main saddles on the tower tops be set shoreward from their final positions a distance of 5 ft. 6 in. in the case of the San Francisco Tower and 3 ft. 7 in. for the Marin Tower. This was accomplished by having temporary brackets on the tower tops to support the overhanging portions of the saddles. Rollers under each saddle permitted the towers to be jacked back toward the shore as the application of the dead-load deflected them toward the main span.

Erecting foot-walk ropes





A large opening was left in each shaft of the Pylons to give room for footwalks and for cable spinning



Since no permanent supports were provided at the splay castings, it was necessary to provide temporary bents to support these in their dead load position until all of this load had been applied.

Cable construction was marked by several innovations in design and erection practice.

Prior to the turning over of the San Francisco Tower to him, the cable contractor had been proceeding with such preparatory field work as could be done without interfering with other contractors. This preliminary work included erection of platforms on both anchorages and platforms and equipment on top of the Marin Tower. While this work at the site was in progress the reeling plant at California City was being constructed and made ready for service. The tower contractor and the cable contractor worked in close cooperation on the top of the San Francisco Tower during the closing days of its erection prior to turning it over to the latter.

WIRE HANDLING: The cable wire was drawn and galvanized in the New Jersey plants of the John A. Roebling's Sons Company, covered with amorphous wax to prevent deterioration of the galvanizing, and shipped by boat to the reeling plant at California City. This plant is located on deep water in San Francisco Bay, which facilitated the delivery from eastern seaports and also shipment of reels of wire to the bridge site. Boats docking at the wharf were speedily unloaded and the coils of wire stored in the 600 ft. x 100 ft. building which had been erected for storage and for housing the reeling plant. A total of 117,000 coils were stored in this manner. A machine and blacksmith shop at the south end of the building was used for maintenance of reeling and cable spinning equipment. All swifting, splicing, de-waxing and reeling to intermediate reels was done on the main floor. A platform raised above the level of this floor was used for reeling from intermediate to big reels. Various movable and fixed overhead cranes were located throughout the mill to handle and move coils and reels.

Wire was received at the plant in coils 5 feet in diameter, 125 to 250 loops, and weighing 200 to 400 pounds. All coils were identified with the official inspection seal and also marked "right" or "left" according to the way they were coiled.

Coils were taken from storage piles to thread-presses, where free ends were cleaned, cut square and threaded for a length of $\frac{7}{8}$ in. at 5500 pounds pressure, the wire being slightly flattened along the thread to prevent rotation in the nipple. After threading, coils were sent to the unwinding swifts according to their marking, dropped over the swift, and the free, front end of the coil spliced to the rear end of the previous coil on that swift. The swifts were horizontal wheels, 5 feet in diameter, with basket frames attached, set on vertical shafts that rotated and unwound the coils when intermediate reelers were set in motion. Tension brakes on each swift prevented coasting when the reeler was stopped. Twenty swifts, set in a V formation, prevented wire leaving one swift from fouling wire from swifts ahead or behind.

The ends of the coils were spliced together with nipples, four nipple presses being located convenient to the swifts. The rear end of the unwound coil was brought to the front or operating end of the nipple-press where it was locked in position between the nipple-press dies. The nipple was then placed a measured distance upon this end. The front end of the following coil was placed in the opposite end of the nipple until the ends butted. The nipple was then pressed on the wire under 5500 pounds pressure, the splice rotated 60° and given an additional press. The splice was then removed from the press and examined. A total of 117,000 splices was made during reeling, of which 11,000 or 9.4% were tested, an average strength of 6820 pounds being obtained. Every hour the inspector on duty would cut out a splice at each press and test it. If a splice failed to meet specifications the press was adjusted until satisfactory splices were produced. Test splices were cut out about 12 inches each side of the nipple for testing.

After leaving the nipple press the wire was run through a hot water de-waxing tank, melting the wax as the wire proceeded, the excess wax being removed by rubber scrubbers. From the de-waxing tank the wire was reeled upon 5-foot diameter inter-

mediate reels, twenty in number, also set in V formation to correspond to the swifts, one swift always being connected to the same reeler. The wire was reeled evenly upon the reels by means of a fleeter that moved across the surface of the reel as the reel rotated. After 10 to 14 coils were put on a reel, it was stopped at a convenient splice, the splice cut out and the full intermediate reel lifted to the big reeling platform. An empty intermediate reel was set in place and reeling again started.

The intermediate reels set upon the platform were placed upon vertical unreeling machines to be wound upon a large reel. Unreeling from the intermediate reels was done by tension exerted by pulling the wire on to the large reels, the intermediate unreeling machine being motor-braked to prevent coasting. Large reels were 6 feet in diameter, 3 feet wide with 9 inch flanges to hold the wire on the reel. The wire was fed uniformly to the reels. Six large reeling machines were installed on the platform, five being in constant use and one in reserve for possible breakdown. As soon as an intermediate reel was emptied, a full one was substituted, the free wire end spliced to the free end on the large reel and reeling continued. Two nipple presses handled all the pressing on the platform. From 41 to 46 coils of wire were reeled upon the large reels. Reeled wire was then covered with waterproof paper and shipped to the bridge site by barge.

Wrapping wire for the cable was handled in the same manner except that the coils were spliced to each other by electric-microwelding instead of nipples. The ends to be welded were gripped in vises and butted together, the microweld adjusted and the welding current passed through the wires, automatically welding the ends.

The welded splice was then removed, clamped in an annealer attached to the welding machine, heated to 1300° F. and allowed to anneal. After annealing, the weld was filed, cleaned, polished and painted and reeled through the de-waxing tank to the intermediate reels.

Erecting the
storm system



FOOTWALKS AND STORM SYSTEM: The footwalk ropes were spread out across the width of the walk so that they effectively supported the transverse planking of redwood without the assistance of heavy cross beams or stringers. To facilitate erection the footwalk was made up in panels about ten feet in length, the transverse boards being nailed to longitudinal spacer boards. Every tenth panel embodied a rigid steel underframe which extended the full width of the walk. Each such frame, consisting of channel cross-beams connected by light floor trusses and braced laterally by diagonal rods, served as the rigid leading panel of a train of ten panels which was erected as a unit by assembling and sliding down the ropes. The rigid panel also provided the stiff cross-members for attaching storm ropes, cross bridges, and the ties between the footwalk and the tramway support ropes. Directly under the main cable the floor of the footwalk consisted of wire mesh.

Overhead, two ropes supported the pipes to which the tramway rope sheaves were affixed. These pipes were spaced at about 200 foot intervals and at each pipe location the tramway support ropes were connected to the footwalk by wire rope ties.

The main footwalk ropes were spliced by means of special sockets near the center of the main span and were discontinuous at the towers and at the splay bents. At the latter points and on either side of the main saddles they were socketed and connected to transverse girders by means of threaded rods which enabled them to be adjusted to the proper sag before the erection of the footbridge panels, and permitted adjustment of the footbridge elevation as the suspended structure was being erected. At the splay bents, a group of seven ropes connected to each end of the transverse girder transmitted the stress to eye-bars embedded in the main anchorage concrete. Similarly, at the main saddles, a group of seven ropes on each end of the girders passed over a temporary saddle and transmitted the pull of the main ropes around the main saddles.

At the center of the main span a structure of considerable weight was required to provide a crosswalk, the framing for the transfer station above the walk, and the storm system connections beneath the footwalk. At four other points in the main span and at one point in each side span, light frames or struts provided the support for intermediate crosswalks.

At the upper-left, the overhead catenary ropes are seen with pipe spreaders on which are mounted the split sheaves which will support the tramway. Below the footwalk are the lower ropes of the storm system.



The storm system was of unique design. The upper part consisted of ropes running beneath the sides of each footwalk and tied to it at about one hundred foot intervals. At the center of the main span these ropes engaged the lower framework of the transfer station. Below this rope and fastened to it was a triangular bracing system which transmitted wind stresses to the towers at two levels—the base of the tower and about halfway between the base and the top. Four struts in the main span tied the vertical planes of bracing together and participated in the stresses induced in the transverse diagonal systems. Thus three more or less distinct systems made up the whole of the storm system: first, the ropes running beneath and approximately parallel to the side of the footwalks; second, the lower diagonal system including the transverse struts; and third, the overhead tramway system with its vertical ties to the walk. The bracing as a whole was very effective against high velocity winds blowing from any direction.

The first step in the erection of the footwalks began in the side spans with the raising of skylines between the anchorages and main tower tops. A traveling carriage supported on these skylines hauled the footwalk ropes from reels mounted at the splay bents, to the tower tops, where connections were made to the transverse footwalk girders. All of the side span footwalk ropes were in this manner pulled to the tower tops (and left in slacked position) before the erection of the main span footwalk ropes began.

In the main span the ropes were unreeled from a barge as it was towed from one tower to the other. The raising of the ropes was accomplished by means of the ropes which had previously been used as skylines for the erection of the side span ropes. The ends of these skyline ropes were passed over the tower tops and carried down to the base

Details of footwalk
construction



of their respective tower legs. When the ends of the rope to be raised had been brought to the bases of the towers, each was clamped to one of the skyline ropes and by them hauled to their respective tower tops. Both ends were raised simultaneously, all operations being controlled by portable radio. Coast Guard cutters stopped all vessels going in and out of the harbor during the raising operation, which lasted about twenty minutes for each rope.

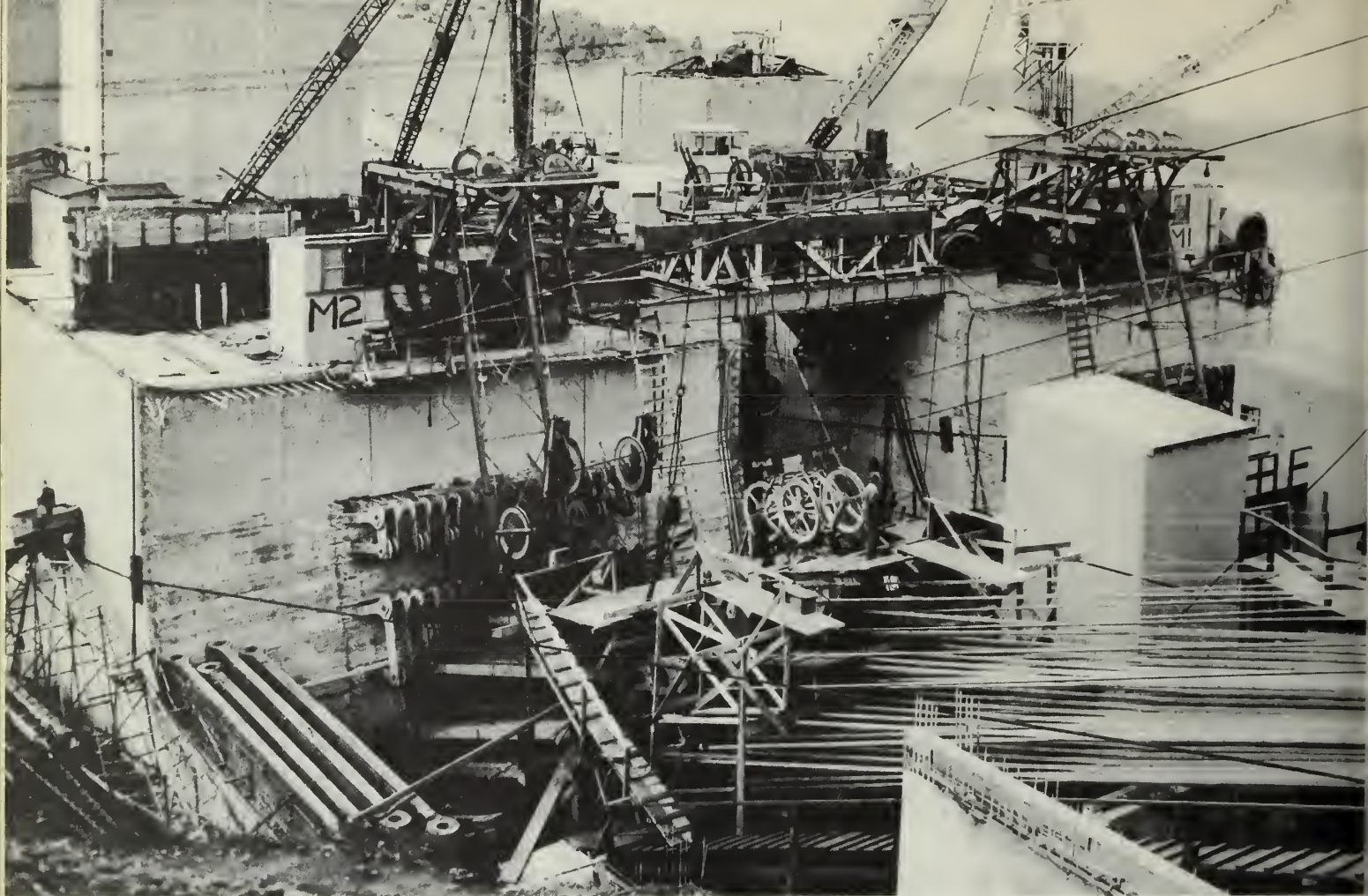
The footwalk decking was placed by assembling the 10-foot long panels into trains about 100 feet in length. The side span trains were assembled at the anchorages and hauled upward, while the main span trains were assembled at the tower tops and slid downward toward the center of the main span.

Due to the exceedingly arduous climbing required of the men who had to move along the footwalks, a man-hauling rope was provided on one footwalk and that footwalk was made slightly wider than the other walk to provide room for the passage of the men who made use of the rope. This rope was placed just above the stationary hand-rail rope and moved at such a rate that the climber could take hold of it and be assisted in climbing the footwalk at a comfortable rate of speed.

SPINNING FACILITIES: Tramway ropes, the function of which was to haul a spinning carriage from the anchorage to the center of the main span and return, traveled upon, and were supported by "split" sheaves, which in turn were supported by the overhead catenary. Return sheaves for the tramway ropes could have been located at midspan except that the installation of the necessary supporting framework would have added materially to the weight already concentrated at that location. For

One of the four-sheave spinning carriages with three bights of wire





The spinning plant at the Marin Anchorage

this reason the cross-over and return sheaves were placed at the farther tower tops. Such a system can be used conveniently only when the spinning program contemplates stringing wire for each cable alternately. Each of the four ropes required was driven by an electrically powered drum located on the anchor blocks and a counterweight was inserted in the anchorage cross-overs for the purpose of maintaining a constant tension on the rope. Each rope was carried over the tower top on two groups of sheaves mounted on frames located above the ends of the saddles. The support for these frames was so designed that the group of sheaves could be shifted laterally to make the spinning carriage pass directly over the strand and live wire sheaves into which the wires were to be deposited, or moved out of the way during the adjustment of the strands.

The principal equipment at each anchorage consisted of the unreeling machines, the wire counterweights, the tramway drive engines, the tramway counterweights, the splice presses, a thirty-ton derrick used principally for handling reels of wire and a ten-ton derrick for assisting in operations carried on in the vicinity of the splay bent. When the spinning operation had developed to where three bights of wire were to be carried by each carriage, six unreeling machines were operated at each anchorage. These machines were hydraulically driven and were capable of rapid acceleration or deceleration at the will of the operator. As the wire was fed from the reels it passed through a counterweight system and the unreeling operator, by observing the position of the

counterweight, speeded up or slowed down the reel to conform to the rate at which the spinning wheel progressed.

On the tower tops a structural frame over each saddle supported the tramway sheaves and the hydraulic lifting jacks which here handled the completed strands. There were two such jacks, of eighty tons capacity, provided on each tower top and these were shifted to either cable as required by the strand adjusting operations. Rollers above the jacks permitted them to be shifted laterally for moving the strands from the spinning saddles to their positions in the permanent saddles. At their lower ends, the jacks were connected to a curved lifting girder which transmitted the pull of the jacks to the strand through a series of special grommets and lifting bands.



The spinning carriage has just passed over the top of tower with three bights of wire (6 wires)

STRAND ARRANGEMENT: Heretofore, all large bridge cables of parallel wire have been constructed with the original hexagon (which makes up the shape of the cable cross-section before compacting) placed with a straight side at the bottom. In other words, the strands were placed in the cable so as to lie in horizontal and diagonal rows. The Golden Gate cables were constructed with a corner of the hexagon at the bottom and with two sides vertical so that the strands lay in vertical and diagonal rows. There were several advantages derived from this arrangement for these very large cables.

With the conventional arrangement of flat bottom hexagon, serious difficulties are encountered at the saddles. Since it is practically impossible to place strands in the saddle and preserve their circular cross-section, the horizontal saddle-dimension of any row of strands must be increased with a corresponding decrease in vertical dimension. Another difficulty is that it is impracticable to hold fillers on the top and center of the strands in the row below that in which the strands are being placed. If these fillers are omitted, there remain voids into which the wires ultimately slip causing the cable to slump in the saddle. This action might not become complete until after the entire load is on the structure and thus bring about a condition of higher stress in the lower wires of the cable. The strand arrangement adopted has a further advantage in that the operation of compacting to a circular cross-section is facilitated because there is involved a minimum vertical displacement of wires.

With the hexagon rotated it is not necessary to try to force a certain number of strands into a given horizontal row and the fillers are much more easily held near their correct location. The new arrangement of strands worked out satisfactorily. No fillers had to be omitted and the strands were, at all stages of construction, very near to their correct positions in the saddle.

Another distinct advantage gained from the chosen orientation of the hexagon was derived from the fact that during construction it was possible to keep separators between the vertical rows of strands at intervals along the length of cable, and to use substantial cable formers which preserved the shape of the cable in spite of non-uniform temperature conditions. The separation between the rows provided ventilation tending to keep the various strands at a more uniform temperature during the day and resulted in a more rapid equalization of temperature at night. Only one difficulty of any consequence was encountered with this arrangement of the hexagon. When only a few strands had been completed and the formers could not be used, no strand was supported directly from the side by any other strand, so that a very slight wind caused the strands to override one another. This made adjustment difficult until sufficient strands were in place to support the relatively heavy formers.

To further facilitate compacting to a circular cross-section, certain strands at or near the corners of the hexagon were made smaller. Although this expedient was of some value in helping to round up the cable it had several disadvantages from a practical standpoint. There was a certain loss of economy in the design of the anchorages; an increase in the cost of erection due to the varying size of spinning saddles, compacting

Cable formers with vertical separators kept strands in vertical rows



tongs and other tools required; and some small nuisances in the planning and execution of strand adjustment. All observations on strand adjustments and much of the adjusting were done during the early morning hours to secure uniform temperature conditions.

SPINNING THE CABLES: The setting of the first or guide wire, required special accuracy and care. Four wires to serve as guide wires were hauled across the west walk from the Marin anchorage and one was selected for the sag observations. In the main span, a levelman located in the west leg of the San Francisco Tower foresighted through a handhole to a lighted target located in the same corresponding handhole in the Marin Tower. He then directed the setting of an inverted rod under the guide wire at the center of the main span. The instrument man directed the movements of the rod target by telephone. Telephones were used also to convey to the center of the main span, the tower verticality readings from the bases of the tower legs, and thermometer readings from the tower tops. When the data from the observations had been assembled at mid-span and interpreted by means of the sag charts, the required movement of the wire was made at the tower top to correct the sag.

The sags for side span guide wires were observed by means of transits located on the pylons at the ends of the side spans. A target located near the tower top served as a foresight to establish a line of sight which was parallel to the side span chord. The distance between the guide wire and the line of sight was measured by an inverted rod held on the guide wire. As in the case of the main span adjustment, the tower leg was plumbed with a transit and temperature readings were taken at three points along the span.

After the first wire on the west walk had been set, the other three were adjusted to identical sags. Two of these were then removed to the east walk, and two additional ones hauled out on each walk and adjusted to the same length as the original four.

The procedure for setting the first strand of the west cable was identical to that followed in setting the guide wire, but for setting the first strand of the east cable it was felt that it was essential to secure an accurate relative elevation between the two cables rather than make an independent setting. In order to accomplish this in the simplest manner, the main span sags were compared by setting up a level on one of the footwalks near midspan and determining the difference in elevation. The difference in span for the two strands was found by making a direct determination of the skew between the saddles on each tower top.

For the side span setting of the first strand of the east cable, the same logic governed the procedure. The difference in spans between the east and west sides was again found by means of the transit on the tower top. The sags for both the east and west sides were observed simultaneously by means of transits at the pylons. The application of the sag-span ratio gave the correct difference in sags to which the east cable had to be adjusted. This method of direct comparison eliminated dependence upon two of the most troublesome observations, namely, temperature and tower verticality.



The transfer station
at the center of the
Main Span

The most radical departure from previous procedure was made in the stringing method. Formerly the wires were strung in each cable by the use of two wheels, each carrying one bight of wire so that four wires were laid in the cable each trip of the two wheels. In this case, however, four traveling carriages were used, each provided (at first) with two sheaves to carry two bights of wire. Each carriage traveled only to the center of the main span where it met another bringing wire from the opposite anchorage. The bights were then transferred from one carriage to the other and the carriages returned to the respective anchorages from which they started. By means of this system, it was possible to have four carriages stringing wire at one time, with sixteen wires strung in a cycle, as compared with four wires by any previous system. The workmen became so proficient in adjusting and putting away the wires that it was later found practicable to add a third wheel to each carriage and pull six wires instead of four so that production was speeded up almost six times over what would have been possible if the methods used on previous bridges had been employed. Each carriage was propelled by an independent hauling rope so that in case of a mishap to one tramway, it was not necessary for all wheels to stop. Frequently two carriages worked while repairs were made on a breakdown affecting the other two.

Wire stringing was conducted alternately on each walk. When the maximum efficiency had been attained, three days of stringing was required to complete one set-up of four strands. Likewise, this was about the minimum time required to put the strands away in the saddles, adjust, and prepare for a new set-up on the opposite walk. Thus the two sets of operations dovetailed very satisfactorily and justified thoroughly the contractor's programming and plant layout based on such a plan. The essential departure from former systems and which made such a program possible, was that four carriages could be operated on one walk instead of two carriages on each of the two walks. Under this arrangement it is noteworthy that the four carriages operated continuously without interruption for strand adjustment, etc., and the same spinning

and adjusting crews were employed continuously on the same work, being shifted from one cable to the other.

At the beginning of stringing, when each carriage carried two bights of wire, two wheels or sheaves were mounted side by side on either side of the carriage. When the three-bight system was instituted another wheel was mounted both fore and aft, and in the plane of the hauling rope, making four wheels on each carriage. The rear wheel, as determined by the direction of travel of the carriage, was utilized to pull the third bight and the front one did not operate. The resultant pull of the three bights was thus kept in the plane of the hauling rope. A total of 42 strands of the 122 was strung by the two-bight system and the remainder, or 80, by the three-bight system. No fundamentally new methods were developed for the unreeling of the wire. The unreeling machines and their operation in conjunction with the wire counterweight towers were not new developments, but had been used before. The stringing of two and later three wires, rove simultaneously on each strand shoe and carriage, was an innovation. On a typical trip, these carriages, traveling at a maximum rate of 650 feet per minute, would progress from the anchorage to the midspan transfer station in about six and one-half minutes. At this point, the bights of wire would be transferred from one carriage to the other. This was a manual operation requiring one minute. The transfer having been accomplished, the direction of travel of the carriages was reversed and they returned to the anchorages from which they started.

The adjustment of the "dead" or standing wires in any given span followed closely after these wires had been laid in the most remote support of that span and the "live" or running wires were adjusted later in the proper span sequence. It was essential in order to avoid confusion, that each of the three wires be properly placed in all live wire sheaves and dead wire hooks and that they be adjusted in proper sequence. To insure this, quick drying lacquer was sprayed on the running reels at the anchorages, one of the wires being thus spotted red, another green, and the third left unspotted.

The motive power for adjusting the wires was provided by moving ropes placed on the walk. These ropes, placed side by side to form a group of four at the end of each span, extended about 100 feet along the walk. They were actuated by electrically driven drums placed be-



Transferring wire from one spinning carriage to the other at the center of the Main Span. The spinning carriages shown have only 2 sheaves each. Later, 2 more sheaves were added



neath the walk, the machines being remotely controlled by the adjusters at midspan. On receiving an electric light signal from midspan, a workman connected the proper wire to a come-along rope by means of a link consisting of a short wire rope with a come-along gripping device attached to either end. The operation was then in the hands of the adjuster at the center of the span. By the manipulation of his control buttons, he pulled-up or slacked-off the wire until it had attained the proper sag. An electric light signal to the clamp-man then instructed the latter to make the wire fast. Because of the length of the main span and the prevalence of fogs, each of the four adjusters was in telephonic communication at all times with the two quarter-point adjusters.

The first twelve or eighteen wires in any strand were adjusted parallel to a guide wire after which the guide wire was removed. All strands were opened up after completion to check the quality of the adjustment. Any wire that fell outside the tolerance for uniformity of sag was corrected by cutting and splicing to the correct length. However, very few splices were necessary. The specifications required that with all wires hanging free, not more than six wires of the entire strand should lie outside of a group measuring vertically seven inches from top to bottom.

Splicing together the ends of successive reels of wire at the anchorages was accomplished by means of hydraulic presses which compressed a steel nipple tightly over the previously swedged ends of the two wires to be joined. For occasional splices which were necessary at points along the footwalk, the common die cut threads and screw nipples were used.

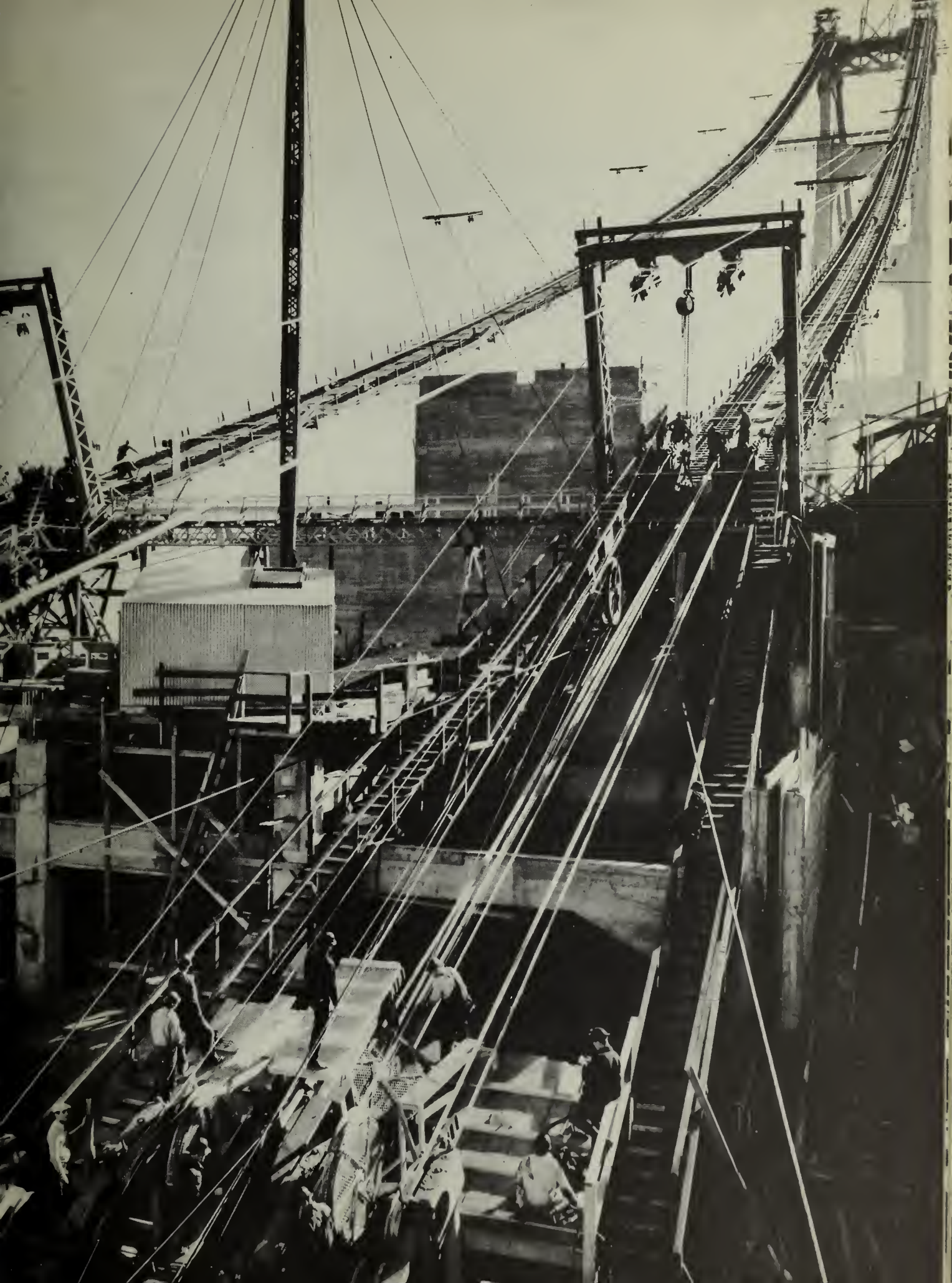
After the spinning of a set of strands had been completed and they had been opened up for inspection, they were seized at five foot intervals by a wrapping of about three turns of heavy adhesive tape. Below the splay point at the anchorages, wire seizings were used. The strands were lifted from the spinning saddles at the tower tops by means of a pair of hydraulic pulling jacks attached near either end of the curved lifting girder. Connection between the strand and girder was made by means of a series of galvanized lifting straps which, with some intermediate seizing straps, had been placed in the spinning saddles before starting to lay wires in the strand. After all of the wires had been placed in a strand, these straps were bent up and connected to the lifting girder through a turnbuckle which provided for adjusting the straps to a uniform tension. The intermediate seizing bands were bent up over the strands and tightened by means of a special device operated by an ordinary hand wrench. The strand was then lifted from the spinning saddle, shifted laterally and lowered into the correct position in the main saddle. When movements of the strand were no longer required, the projecting portions of the lifting and seizing straps were cut off, the inaccessible portions being left beneath the strand. At the splay bents, where the deflection of the cable was slight and the spinning saddle assembly was comparatively short, the strand was seized with wire and lifted by means of rope grommets attached to a short lifting beam.

During spinning, the strand shoe at the anchorage was mounted ahead of its final position on a cast nickel-steel strand leg which held the strand shoe so as to leave one side



Above, left: Lifting a strand. Above, right: Lifting straps and seizing bands were cut away after the strands had been finally placed in the saddles. Below: Preparing to lift a completed strand out of its position in the spinning saddles. Opposite page: In the foreground, strand shoes are shown just forward of the eye-bars, in position to receive the wire



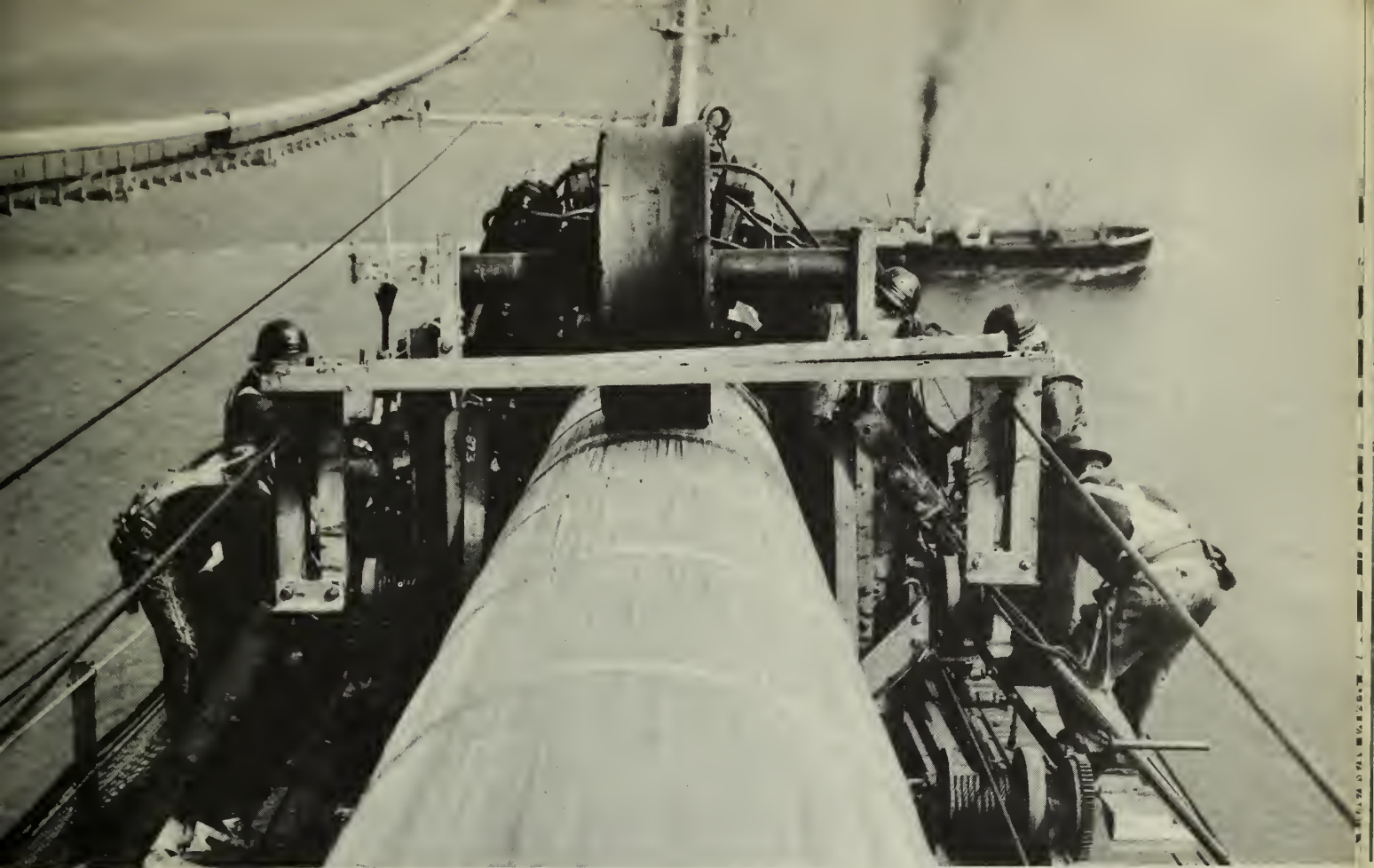


unobstructed for the placing of the loops of wire. After completion of the placing of wires in a strand, the strand leg provided the link between the strand shoe and the pulling-jack mechanism by means of which the end of the strand was pulled back to its correct position between the eye-bar heads.

For adjusting a typical set-up of four strands, movements in the main span were made by shifting two of them over each tower top. In the side span those strands which required no shifting at the adjacent tower top were adjusted first by inserting shims in the strand shoes at the anchorage. After two strands had been satisfactorily adjusted in the main span they in turn were set in the side span. It appears that the ideal program for adjusting strands is to attempt to set them slightly high during the daytime, take readings that same night and make the adjustments the following day, basing the movements at the saddles and the shim changes on values computed from the night readings. This method is satisfactory provided that the differential sag-length ratios are accurately known, and provided the computations and movements are carefully supervised. To insure correct results, however, the settings were checked on a succeeding night and further changes made if then found necessary. Obviously this method required a considerable length of time and, where delay might impede spinning on the next set of strands, it was more desirable to bring out the adjusting crews at night and complete the operation in a single shift. Each of these methods was used at different times on the Golden Gate Bridge. For the first few set-ups, the crews worked at night until the accuracy of the ratios was verified. For subsequent set-ups the latter method was used except where the contractor elected to adjust at night to avoid possible interruption of the spinning. In all, 52 of the 122 strands of the two cables were adjusted by reading at night and making the necessary movements during the following day; the remainder, or 70 strands, were adjusted by night crews.

The correct position of a strand was not determined by tangency with the strand beneath, but by a calculated dimension from the under frame of the cable former, located at the center of each span. Throughout all adjusting operations, an effort was made to avoid a flat cable. For instance, the side span strands were pulled up from one-sixteenth to one-eighth of an inch farther than the necessary sag change would indicate after experience had shown that this closely corrected the slight drop due to slumping when the seizing bands in the tower saddles were cut.

COMPACTING AND WRAPPING: Six compactors were used to squeeze the cable to a circular cross-section. Each consisted of a frame which surrounded the cable and supported a battery of twelve jacks radially directed so as to exert a force normal to the surface of the cable. The jack assembly was supported on a structural steel frame which was designed to travel on the cable on large wooden spools. The frame was stationary for three moves of the jacking assembly which rode on tracks on the frame. After the three moves, the jacking assembly gripped the cable, and the supporting frame was moved ahead to a new position in readiness for three more moves of the jack assembly.



One of the six compacting machines

A close-up showing radial hydraulic jacks of the compacting machine. There are a total of twelve jacks in the complete circle



All movements of the assembly were made by means of hand-operated cable winches. Each of the jacks having a piston diameter of $5\frac{7}{8}$ inches and operating under a maximum pressure of 6000 pounds per square inch, exerted a pressure of 81 tons, or a total of 972 tons for the twelve jacks on each compactor. The shoes contacting the surface of the cable were 6 inches wide and completely surrounded the cable so that the total contact area was about 678 square inches. Assuming 10% friction-loss in compacting, the resulting maximum unit pressure per square inch of contact surface would be 2580 pounds. After the start of operations, it was found that little further compaction was accomplished after the fluid pressure in the jacks had exceeded 5000 pounds per square inch and, for most of the work, this pressure was used. An electrically driven pump for supplying the fluid pressure accompanied each compactor, being moved along the foot-bridge on a timber sled. Incorporated in the compacting unit were two auxiliary hydraulic jacks which tightened the seizing bands. The cable was compressed at intervals of three feet, which therefore was, in general, the seizing band spacing. The seizings consisted of galvanized metal straps, two inches wide, which were held tightly by engaging the ends in a special socket and wedge device. After the adjustments under working conditions had been made, each compacting machine was able to compact 204 lineal feet of cable per day of eight hours.

The mean diameter after compacting was close to $36\frac{1}{16}$ inches (at the seizing bands). Under the permanent bands, after final tightening, the mean diameter was $35\frac{7}{8}$ inches.



As soon as one whole span of either cable was compacted, it was immediately measured for length. All measurements were conducted at night and by two independent field parties. The band spacings, as previously computed on the basis of theoretical lengths, were then corrected to distribute the difference found between actual and theoretical length, which was small. For securing the correct location of the vertical axis of the bands, a true bottom of the cable was located at night after the cable had assumed a uniform temperature throughout its cross-section.

All cable bands were lowered from the tower tops by means of carriages riding on the ropes which had supported the tramway during the spinning operations. For the placing of cable bands and suspenders, and the jumping of the wrapping machines, these ropes served as high-lines to support carriages upon which were mounted chain-falls for lifting and transporting the moderate loads above the cables.

The band-bolts were tightened to a high stress almost immediately upon erection of the band. The specifications called for a stress of 92,000 pounds in each bolt. In anticipation of a diminishing stress with the addition of dead load, the bolts were tightened at the first tightening to approximately 120,000 pounds each. The final tightening was done when a little more than half of the concrete had been placed on the roadway and at this time the bolt stresses were found to have slacked off to between 70,000 and 80,000 pounds. For the final tightening, the stress was again brought up to 120,000 pounds to allow for the effect of the remainder of the dead load yet to be added.

Various types of wrenches for tightening the bolts were tried. An electrically driven gear and ratchet wrench had been developed by the contractor for this particular purpose and was used for the first tightening. A wheel wrench 5 feet in diameter, which was actuated by a cable wrapped around the grooved rim and pulled by an electric hoist was used later. A short socket wrench, turned by striking the end of the handle, was used for the final tightening.

All of the bolts were tapped at both ends with a small conical hole for the application of a strain gage. A record was kept of the strain readings on all of the bolts at various stages of the operations. Special extensometers were designed for taking these strain measurements.

The suspenders consist of four parts of 2 11/16 inch diameter wire rope. The longest suspender rope is approximately 980 feet long overall. They were unreeled from the tower tops, both ends being hauled down the footbridge by the use of the high-line, and then lowered over a pair of temporary sheaves mounted on the cable band. A center mark on each rope was made to correspond with the top of the split in the cable band.

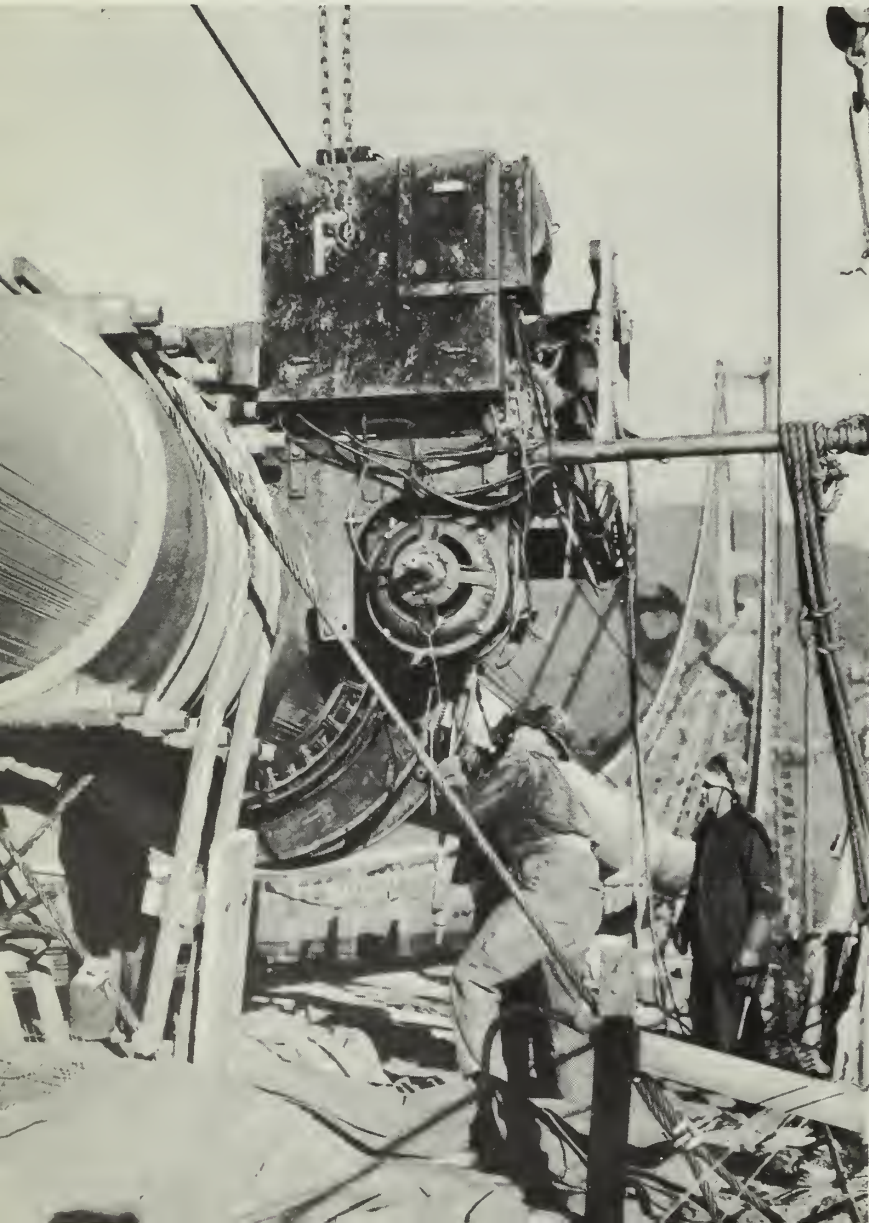
Six wrapping machines were used on the cable wrapping. These machines were of a type developed by the contractor erecting the cable. In this type of machine, the bobbins are so constructed that the wire on them surrounds the cable. Two wires are wound on the cable simultaneously so that two bobbins are provided on each machine. With this type of machine, the wrapping must stop while the bobbins are being filled. The wrapping wire was fed to the bobbins from reels mounted on the tower tops. The machines

were designed to function either as "pushers" or "pullers." This means that they could be operated either to ride on the bare cable with the wrapping wire being laid *behind* the machine, or to ride on the wrapped cable with the wire being laid *ahead* of the machine. This adaptability was utilized to eliminate most of the hand wrapping usually required near the cable bands.

The machine was assisted in its progress by means of the tension on a rope passing over a sheave at the tower and carrying a heavy counterweight. The advance was controlled by a tie-back set of falls released by a hand winch. A workman, closely observing the laying of the wires, released the tie-back cable at the required rate of speed and permitted the forward cable to advance the machine. After the initial running adjustments were made and the crews trained, the wrapping machines were daily wrapping two and one-half 50-foot panels of cable "pushing" and four and one-third 50-foot panels of cable "pulling."

All splices in the wrapping wire were made by means of portable electric welding machines which were carried along the footwalk with the wrapping machines. These

One of the six
wrapping machines



welders were especially designed for this purpose and besides producing a butt weld, they also, by a second operation, accomplished the annealing necessary when welding wire of high carbon content.

Since a considerable length of time was to elapse between the completion of the compacting and the cable wrapping, the cable was given a coat of Galvanized Metal Primer immediately after compacting. The wrapping wires were laid in a heavy red lead paste applied immediately ahead of the wrapping machine. Over the wrapping (and on the suspenders) another coat of Galvanized Metal Primer was applied, followed by a paint containing 68% red lead and 32% glyptol type synthetic vehicle. As a final covering, the cable and suspenders were given a coat of International Orange Paint.

ERECTION OF STIFFENING TRUSSES AND FLOOR

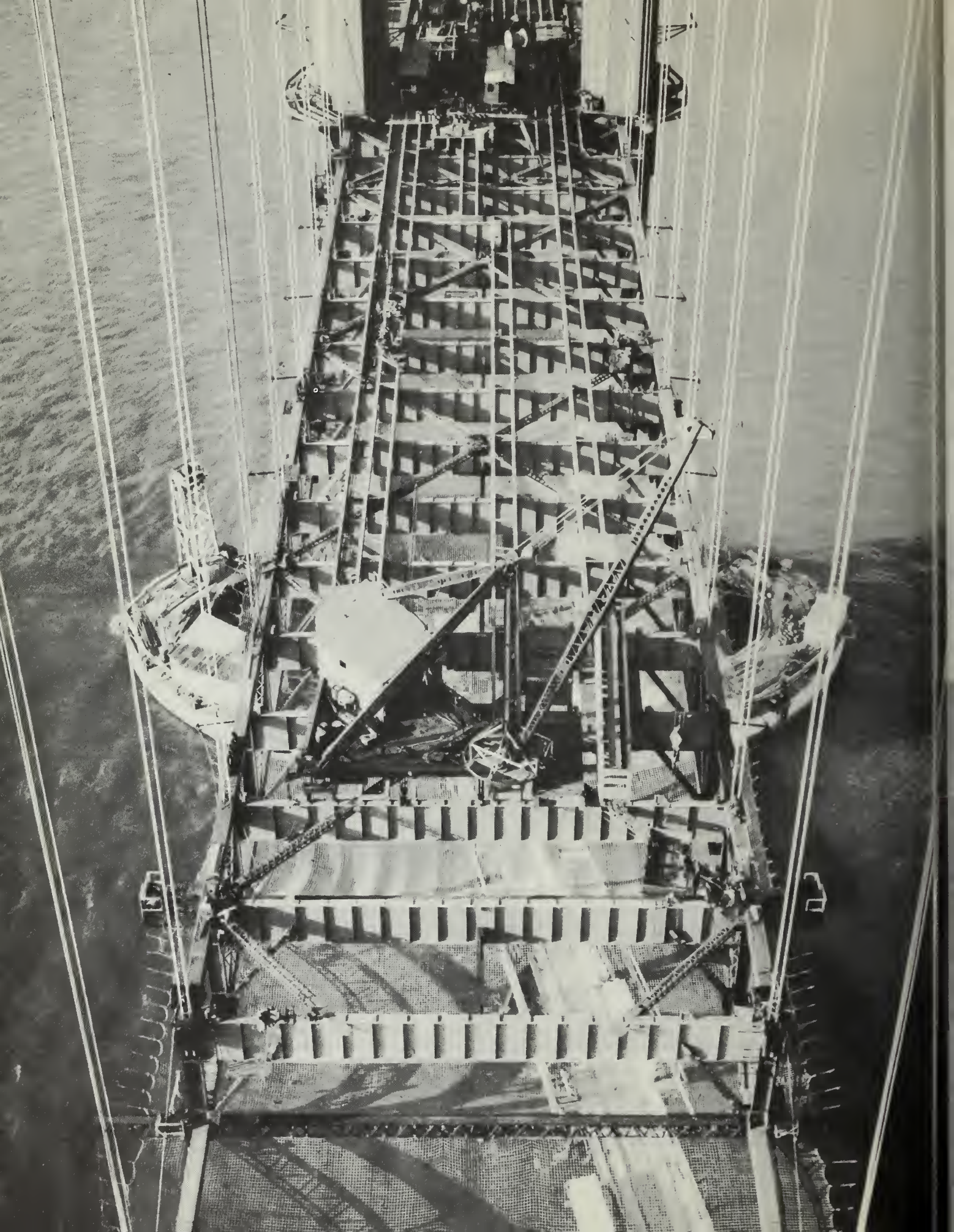
The erection of the stiffening trusses and floor on the Golden Gate Bridge was given thorough study. Shipments of steel from the East were scheduled to arrive not only well in advance of the time they were needed for erection but also prior to the date of an anticipated shipping strike. The material was unloaded from the ocean going vessels at the Alameda plant of the Bethlehem Steel Co. At the yard, the various pieces were sorted and stored in the order they would be required for erection. Moreover, an inspection of the painted surfaces was made by the Engineer and a definite painting program adopted, as described below.

All material for the suspended structure under the specifications, was intended to have one shop coat of red lead mixed in the proportion of 25 pounds of red lead to a gallon of linseed oil. From experience with the tower steel, it was determined that steel shipped from the east by boats should have greater protection than that afforded by the one red lead shop coat. As a result of exposure tests of paint at the site, it appeared that the best paint for the Golden Gate Bridge was quick drying material composed of pigment and a synthetic vehicle of the glyceryl phthalate type. Although a portion of the suspended structural steel had already been shipped with the original shop coat applied, it was decided to change the specifications so as to permit the application of not only the priming coat, but also the second coat (and in some instances the third) of paint in the shop as an additional protection for the steel during shipment. Based on the experience of the exposure tests a synthetic red lead paint as described above was used for the three under coats of paint. A careful study was made to determine what surfaces could be given all three coats and what surfaces only part of them. In general, the second coat was applied to all surfaces and the third coat was applied to the insides of boxed sections. All of the material after unloading from the ocean going steamers at the Alameda yard of the contractor, was carefully inspected and any damage to the paint coats was repaired. In many cases, this required entire re-painting of certain material that had been shipped prior to the change in paint procedure. It was intended that before any piece of steel left the storage yard for the site, the paint should be in such shape as to meet with the revised specifications as to the number of coats in good condition on the various surfaces.

The supplementary painting agreement included the application of one coat of the final paint to the outside surfaces of both steel towers and this was applied prior to commencement of cable erection.

As required for erection, the steel was loaded on barges and towed to the site. Here it was unloaded onto a material platform at the base of each tower, using two one-hundred foot Chicago Booms at the roadway level for this purpose. The various pieces were lifted to the roadway level by the Chicago Booms, loaded on flat cars and hauled out to the travelers as the work progressed.

The time required for erection of the suspended structure was one of the factors

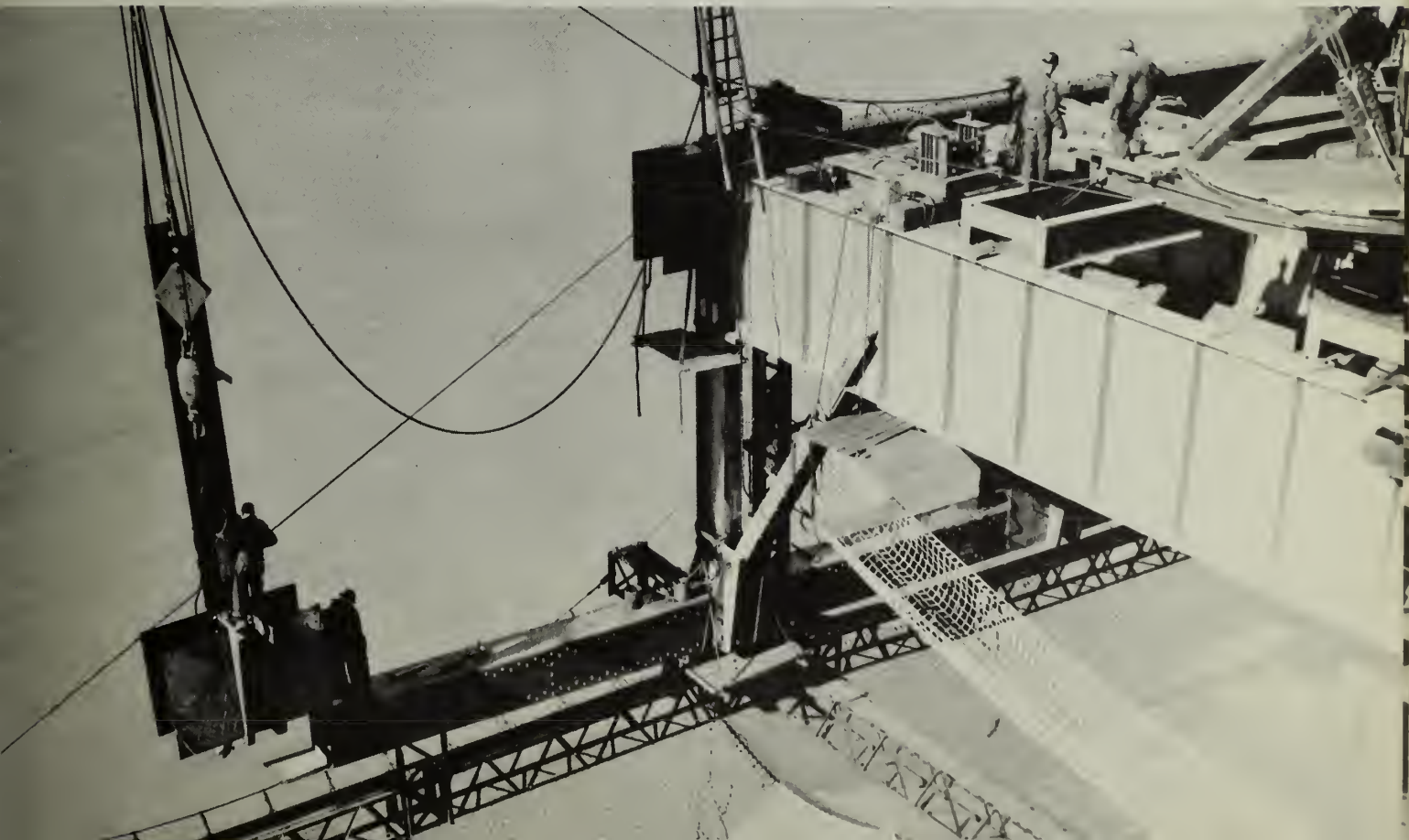


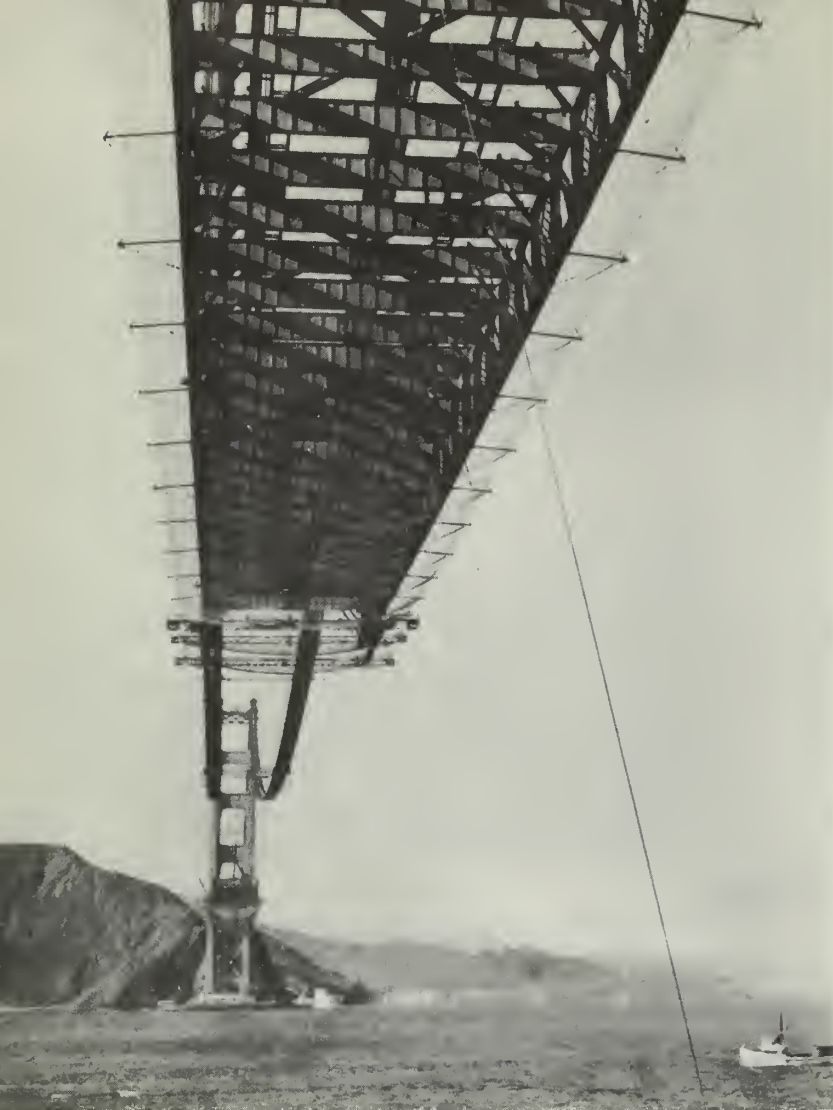
which determined the date of completion and therefore, it was important that the time be minimized by adopting a carefully worked out procedure which took cognizance of all special problems. This procedure was worked out in detail in advance of the commencement of the work so that it was determined in advance what fitting up and riveting could be accomplished at the various stages of erection. The benefit of this well studied program was reflected in the rate of progress effected.

A great aid to the speed of erection was the use of a safety net. This net made of manila rope, $\frac{3}{8}$ in. diameter and 6 in. square mesh, was placed progressively under the suspended structure as the latter was erected so that eventually it extended under the bridge its entire length between pylons and was wide enough to extend ten feet outside the trusses on both sides. This safeguard was required by the Chief Engineer. In addition to saving nineteen lives during construction, there is no question but what the men worked faster and more efficiently since they felt the protection of the net below them and were able to move about more freely. The use of the safety net has attracted only favorable comment from the public and has set a precedent that will long be remembered.

The procedure adopted, permitted the suspended steel erector to start work before the first suspenders were ready to receive the loads. This was accomplished by cantilevering the first three panels of trusses and floor out each way from the towers, thus gaining working space on which the four traveler derricks could be assembled. Each traveler, weighing about 79 tons, was equipped with one 75 foot boom. Power was derived from oil-burning steam boilers.

Left: Erecting the suspended structure. Note the safety net, wider than the bridge, always extended out beyond the point where men are working. Below: The net at forward end of erection was carried on a traveling crane which cantilevered out ahead of the workmen. When it moved forward it paid out a section of net behind itself





Erection of the suspended structure began on June 18. On July 20, 1936, all four travelers were in place and rigged up ready to proceed with the erection except that the safety nets ordered by the Engineer had not been provided. Owing to delay in the manufacture of cable bands by the Roebling Company, the placing of the suspenders near the tower first required for continuing the erection of the stiffening trusses, was not accomplished until August 31, 1936. Erection of the stiffening trusses outward from the initial cantilever erection, started on September 11, 1936.

The general procedure of erection of the first-pass steel was to cantilever out an additional two panels of truss from the last connected suspender. The traveler then raised the outward end of the truss until the next suspenders could be connected to the newly erected vertical. As soon as the suspenders were connected, the traveler slacked off the truss until its weight was taken by the newly

connected suspenders. A similar operation connected the opposite truss members. With the new two panels of trusses connected to the suspenders, the floor-beams and laterals were erected and the necessary stringers to move the traveler forward were placed.

When the travelers met at the center of the main span, the closing top chord members were placed. Following this, the closing bottom chord members were placed. Since under this condition, the distance between bottom chord panel points in the closing panel would be less than normal, in order to provide for entering the last bottom chord with at least 1 inch clearance, this member was fabricated $3\frac{7}{8}$ inches shorter than normal. With the top chord joints pinned for a stress of not less than 700,000 pounds, the bottom chord panel points of the closing panel could be jacked apart and the closing chord pinned in place. Thus the stiffening trusses were made continuous from tower to tower with proper configuration.

During the entire operation, the cable contractor manipulated the jacks behind the saddles at the tops of the towers, jacking the tower tops shoreward in accordance with the prescribed program which kept the tower deflection within 12 inches in either direction. As the travelers moved out in all spans from the towers, the structure

assumed a curve that was concave upward. Under such a shape, the top chords were in compression and the milled ends tight together. While in this condition, the top plates of the top chord splices were riveted. Later, when the trusses assumed their normal shape, (convex upwards), these riveted top plates acted as a tie to prevent the opening of the top chord splices when the bending moment induced in the trusses caused the bottom chords to be in compression. The bottom chord splices consequently came to a bearing under this load condition. As a result of this manipulation, all milled ends of chords came to proper bearing. It was not permissible to rivet chord splices unless the opening between milled ends was .006 in. or less. Since the chords were in compression at the time of riveting, even if the joint was open by the amount of the allowance, as soon as riveting was started the jarring and heat caused the opening to close.

As rapidly as the erection of the first pass proceeded, the lateral system was riveted. This was possible since the lateral system is in the plane of the top chord and from the beginning, the top chord joints were tight. Where the riveting gangs were not able to keep up with the erection, the lateral connections were completely pinned and bolted. As a result of this procedure, the lateral system was connected up soon after the trusses were joined in the center and thus the lateral stability of the trusses under erection condition was early achieved.

Erection of the second pass material, which included the remainder of the roadway stringers, sidewalk material and curbs, was effected as the travelers moved back toward the towers. The steel curbs were set in position and aligned, then removed and set aside to clear the operations of the paving contractor.

As soon as the material had been erected, all field seams between the gussets and the main material were given two coats of paint prior to the general application of the third under coat. Field rivets and gusset plates were treated likewise. Any damaged surfaces were spotted with three coats of paint. The intent being that with the completion of the steel erection, the surfaces would be ready for the final coat of paint and any damage done by other contractors would be their responsibility for repairs.

Seven lines of stringers were placed during the first-pass of the travelers. The balance of the stringers together with the curbs and handrailing were placed during the second-pass. Note painters on the suspenders



The first-pass erection was completed when the two center span travelers met at mid-span on November 20, 1936. Substantially all suspended steel was erected on December 14, 1936.

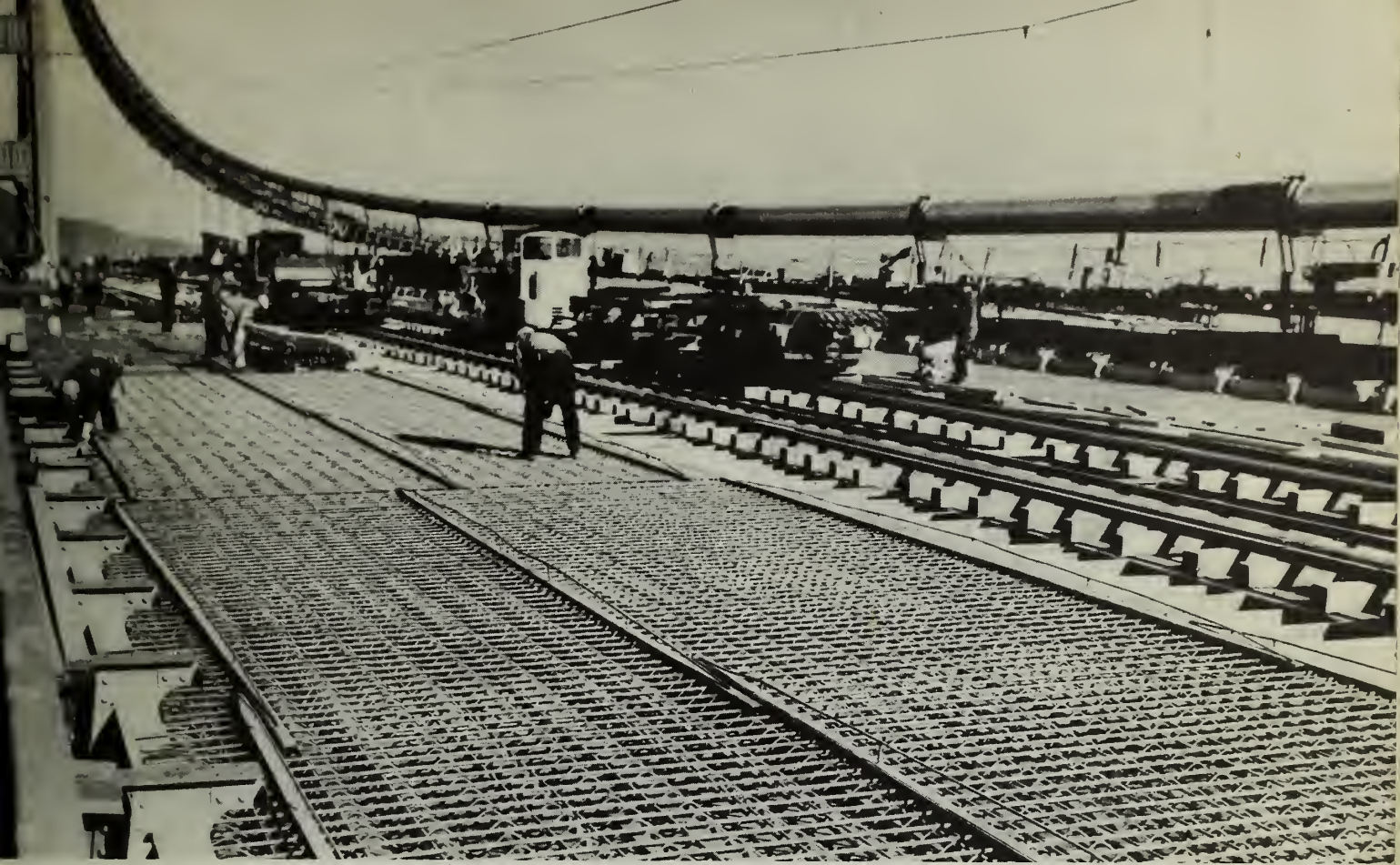
ROADWAY PAVEMENT AND SIDEWALKS

A carefully worked out program for placing the concrete roadway was incorporated in the paving contract. This program took advantage of the fact that as all travelers moved back toward the towers they left behind them the steel floor-structure ready to receive the concrete pavement. There was room at one side for the paving contractor to operate a material track past the travelers. The specified program, therefore, required the paving contractor to follow immediately behind the travelers, as they moved back toward the towers, placing forms and reinforcing steel for two outside, 20 ft.-wide strips of pavement. The material for forms and reinforcing steel for the main span was to be taken out over a material track at one side. An understanding had been reached with the steel contractor that in return for his cooperation, the District would consider that he had met the essential time requirements of his contract when the paving contractor was thus able to proceed with his work. Under this program when the travelers reached the towers, the paving contractor would have completed the placing of forms and reinforcing steel in two 20 ft.-wide strips throughout the length of the bridge ready for concreting. Unfortunately, due to lack of cooperation on the part of the contractors, the three weeks time which could have been gained by this program was lost and no effective work was done at the center of the main span until the travelers reached the towers and were dismantled.

On January 19, 1937, 34 days after the placing of steel had been completed, the first concrete in the roadway deck was placed.

The roadway slab was divided into three longitudinal strips, 20 feet wide for ease in finishing and construction. It was divided every fifty feet with a special pre-moulded expansion joint to prevent the participation of the roadway slab in the truss stresses. All pours therefore, were in widths of 20 feet and in multiples of 50 feet in length. Wooden forms were used, suspended from the tops of the stringers by special supporting devices that were removed with the stripping operation. The slab was reinforced transversely with fabricated reinforcing steel trusses having the web members of the trusses arc-welded to their chords by machine-made welds. Longitudinally, the slab was reinforced with $\frac{1}{2}$ inch round bars. The trusses were held the required distance above the forms (1 inch clear) by steel blocks which were welded both to the trusses and to the tops of the stringers. The field welding was done by portable gasoline driven welding outfits. Form panels from the roadway were re-used in the sidewalk construction.

Concrete was hauled over the construction trestle to the base of the San Francisco Tower in transit-mix trucks. Here it was dumped into a hopper and raised to a hopper above the roadway level by means of the usual concrete hoist. At this point it was loaded



Placing forms and reinforcing steel for bridge pavement

A section of roadway shown in the foreground is ready for concreting



into industrial railway side dump cars and hauled to the point of deposit by gasoline industrial locomotives. At the point of deposit, the concrete was distributed by hand operated buggies. Special vibrating finishing machines were used in addition to hand controlled mechanical vibrators in order to insure a smooth soffit and dense concrete. With the reinforcing steel held solidly in place by welding to the stringers and the top surface finished with mechanical surfacers on the finishing machines, the finished slab was well within the tolerance of $\frac{1}{8}$ inch in 10 feet set up by the specifications. The maximum rate of progress was 1700 lineal feet of 20 foot slab in eight hours.

Concreting of the deck proceeded without serious interruption until February 17. On this date an unfortunate accident occurred which cost the lives of ten men working on a stripping scaffold near the center of the main span. It also held up further operations of the paving contractor. In this accident, the stripping scaffold fell into the net and carried away the entire half of the main span net between the net traveler and the San Francisco Tower. Workmen were not permitted to work in the area where the net



was missing. Materials however, were hauled over this area to the Marin side where construction of forms and setting of reinforcing steel went ahead.

On March 3, 1937, replacement of the net was started and it was completed on April 2, 1937. Concreting operations were immediately resumed, with the result that the roadway paving was completed on April 15, 1937. During this operation the west lane was kept clear for the cable contractor who was engaged in removing the wooden sections and footwalk ropes from the west footwalk. Other operations by the erector were necessary at the same time so that the work of the three contractors had to be carefully coordinated in order to avoid confusion and in the interests of safety.

As fast as the form material could be stripped from the roadway and placed in the sidewalk space, sidewalk concreting was put under way. The speed of this operation depended upon the number of expert finishers employed. Since all operations on the bridge at this time began to feel the scarcity of skilled workmen of all crafts, the rule for employment of residents only was waived where necessary so as to minimize delay.

During the concreting operations of the paving contractor, there was considerable leakage from the forms which splattered the painted surfaces with grout and cement stain. The paving contractor was required under his contract to clean off this stain and repaint wherever the paint coats were damaged by his operations. In general, this was wherever the concrete or cement stain had to be removed since it was necessary to use steel wool and putty knives to clean off the foreign material. These surfaces were carefully cleaned under the supervision of an inspector and then retouched with the corresponding coats of paint to restore them to a condition ready to receive the final paint coat.

CONSTRUCTION OF SAN FRANCISCO AND MARIN APPROACHES

The Marin Approach piers carrying the structure between the abutment and the anchorage were located in various types of ground. The footings on the side of the hill were all in exposed rock of the hillside. Those in the valley were in swampy ground where wooden sheet pile cofferdams were necessary to reach suitable foundation material.

The San Francisco Approach piers were carried down into the sandstone ledge on the hillside and at such elevations that they could not be affected by either the excavations for the anchorage blocks or for the adjacent piers. Excavation for these piers was carried on within the confines of wooden sheet pile cofferdams where necessary.

Erection of the Marin Approach superstructure was started on January 17, 1936. The trusses were erected on specially designed timber falsework towers which were framed with timber connectors. These towers were made up in sections so that they could be used for varying heights of spans and re-used for both this approach and the San Francisco Approach trusses and arch. A stiff leg traveler derrick was used and erection of the Marin Approach was completed on August 21, 1936.

Erection of the San Francisco Approach superstructure was started on July 21, 1936, using the same traveler that had been used on the Marin Approach. Erection of the 319 foot arch over Old Fort Point was carried on simultaneously. For the erection of this two-hinged arch, two guy derricks were utilized for the arch ribs after which the erection was completed with the approach traveler when it had reached that point. With the aid of jacks on the falsework, the chords of the arch were brought to proper bearing.

CONSTRUCTION OF PRESIDIO APPROACH ROAD

The Presidio Approach Road includes the structural steel High Viaduct, the reinforced concrete Low Viaduct, and the paved roadway and grading between the Toll Plaza and the intersection of Lyon Street with Marina Boulevard at the eastern limits of the Presidio. In January, 1934, in order to help relieve unemployment in the District, the contractor was permitted to proceed with the work on the Low Viaduct in advance of the time that it was required. The main contract for this work was awarded in two parts of which Part II contained the Low and High Viaducts. Work on these structures was undertaken only as rapidly as the funds were appropriated by the Board of Directors of the District. Part I of this contract was awarded on August 4, 1936.

The Low Viaduct structure consists mostly of simple reinforced concrete spans of 32.5 feet. The footings are on piles as are the footings of the east abutment for the structure. The length of the structure between abutments is 3308 feet. Wherever the cut-off for the piles was below ground water, wooden piles were used. On the hillside where ground water was at great depth below the surface, concrete piles of the cast-in-place type were used. The length of wooden piles averaged between 38 and 40 feet. All piles were driven to refusal.

Because of the swampy character of the ground under the viaduct, it was necessary to adopt a type of falsework for supporting forms that would not be affected by the variable ground conditions under the structure. Accordingly, all falsework was supported by the foundations of the structure itself. Each column was enclosed with a four post tower bearing on the corners of the footing slabs. This tower supported a falsework of rolled steel beams utilizing the stringers from the High Viaduct and its approaches before they were required for erection. These steel beams supported the forms themselves and proper allowance was made for the deflection of the beams so that the resulting roadway floor would be a true surface as required.

This structure, being composed of alternate four post towers with simple spans between, lent itself to an economical and rapid procedure for construction. In the simple spans, one end was fixed to the tower span with steel dowels and the other end was left free to move by means of metal expansion plates embedded in the concrete. The upper bearing plates were of steel and the lower ones of bronze. On account of the heavy reinforcement of the columns and beams of the viaduct, internal vibrators were used



to insure dense concrete free from voids. Work on the Low Viaduct was started June 18, 1934 and completed on September 22, 1936. The Low Viaduct also provided connections for both ramps of the Richardson Avenue Approach.

The piers for the High Viaduct and the short approaches at each end were originally designed as spread footings to be carried to suitable material. Foundations for Pier 4 and westward were so constructed. Borings at Pier 5 and eastward disclosed, below the sand formation, a stratum of peat of varying thicknesses. It was therefore decided to place the piers located above this stratum on concrete piles. Four piers were thus founded.

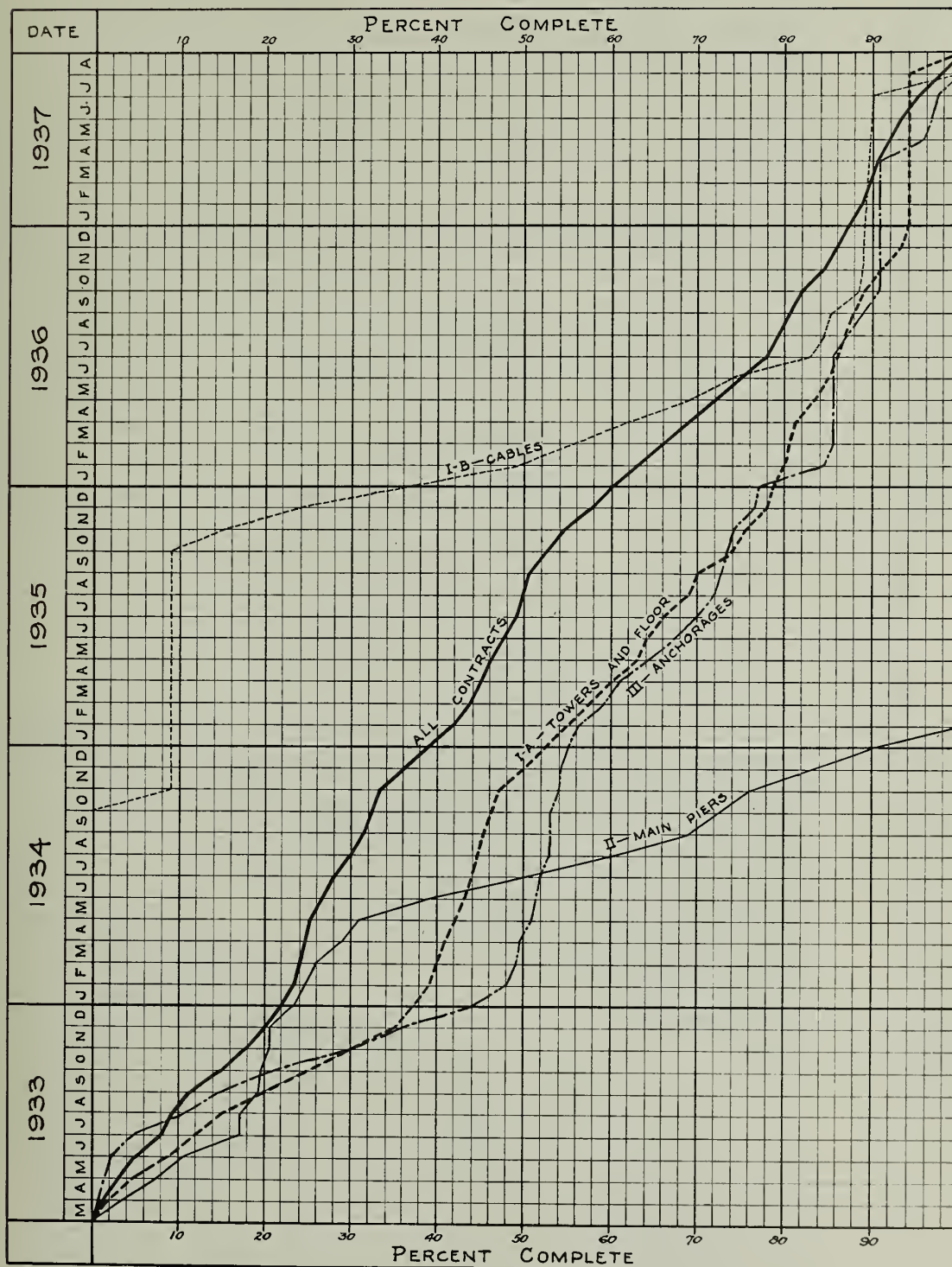
The concrete was deposited in the upper part of these piers by a locomotive crane with dump buckets. Internal vibrators were used to compact the concrete and insure its density. Work on the High Viaduct substructures was started on February 23, 1934 and completed June 5, 1935.

The erection of the High Viaduct superstructure involved no unusual difficulties. The erector did however adopt a unique falsework plan for this work. Clusters of six long piles were driven a few feet into the sand soil beneath the structure until their tops were just below the elevation of the bottom chord. The tops were pulled together and lashed with wire rope. Caps were placed on top of the clusters and the chords supported from them.

Erection of the High Viaduct, together with the concrete roadway slab which was a part of this contract, were completed on December 4, 1936. A simple concrete balustrade was used on both viaducts.

Plywood panel forms were used for the concrete structures.

CONSTRUCTION PROGRESS DIAGRAM





BOOK FOUR

**MATERIALS. MANUFACTURERS
THROUGH EXTENSIVE RESEARCH
AND DEVELOPMENT HAVE PRO-
DUCED MATERIALS OF HIGH STRENGTH
AND QUALITY WITHOUT WHICH THE
WORLD'S LONGEST BRIDGE SPAN
COULD NOT HAVE BEEN BUILT.**

MATERIALS

THE MATERIALS of which the structure is composed vary in class and grade because of the type of the bridge. The basic structural divisions of the bridge consist of towers and anchorages, cables and floor system. The towers and the anchorages sustain the cables while the floor structure is suspended from the latter. The various divisions are composed principally of rolled structural steels which occur in the towers and suspended structure, heat-treated eye-bars which are embedded in the anchorages and wire which forms the cables. The latter have such appurtenances as cast steel bands and suspender ropes of wire while throughout other parts of the structure are distributed various kinds of other materials. With the exception of cable wire, the materials had to be either combined or worked into members at the shop.

The quality of each grade of material, with some exceptions, was ascertained by means of both chemical and physical tests. The chemical determination comprised ladle and check analyses. The physical determination embraced the regular mill or foundry tension and bend tests and the check tension test. The physical check test was made to obtain more accurate information regarding the properties than can be secured from mill tests and was intended primarily for structural steel but was used for all materials.

Application of the tests varied with the character of the material. For every melt a ladle analysis was made on samples secured at the time of pouring the molten metal into molds. A check analysis was made on either semi-finished or finished material. The physical tests were employed in a manner controlled by the type and grade of the material and the method of its manufacture. At one end of the list was material such as wire of which each piece was tested. At the other end was material such as eye-bars of which representative pieces had to be investigated. In between these extremes was the material that was tested by means of random choices of samples.

The different tests were used, of course, for the purpose of gaining assurance that the prescribed properties of each grade of material were within the limits set by specifications. In this connection, three conditions which attend manufacture, viz., chemical composition, mechanical treatment and heat-treatment, had to be considered. These factors are variables. The element carbon, for example, because of conditions under which molten metal solidifies in an ingot mold, tends to segregate in such a manner that the top of the ingot becomes high in carbon content and the bottom low in this respect as compared with the intermediate portion. Concerning mechanical treatment there occur, for instance, many cases where ingots in a melt of steel, and even parts of an ingot, must be worked differently to produce pieces of different thickness and sizes. Thermal variations occur in all the heat treating practices. Of the three factors, composition is the underlying one. The specifications have recognized their variable nature by providing means for investigating their effect. A check analysis was authorized in

connection with material suspected of being unduly segregated. Physical tests besides the usual number were required when the thickness of material exceeded defined limits. And for each lot of heat-treated material physical tests had to be made.

Accordingly, the testing programs were arranged to include specimens from those parts of each lot of material which would reveal the ranges in properties. The chemical properties were found by analysis of material from top and bottom cuts of ingots whenever possible. The physical properties were determined from tests of like material when it was used for the work and when the number of tests required by the specifications permitted such a selection.

In order to acquire the desired information, identification of material naturally became an important consideration. The identity of not only the melt but also of the pieces within the melt had to be known. In the discussion bearing on the various material, the manufacturing procedures are described for the purpose of showing the system of identification used as well as the reasons for adopting the different test methods.

The materials in the main steel structure of the bridge are discussed here under the following headings:

1. STRUCTURAL STEEL
2. HEAT-TREATED EYE-BARS
3. WIRES AND ROPES
4. CAST STEEL
5. FORGED STEEL
6. OTHER MATERIALS
7. DISTRIBUTION OF MATERIALS IN MAIN STEEL STRUCTURE

STRUCTURAL STEEL

The principal grades of rolled steel in the bridge are silicon, carbon and rivet. Included under carbon there is copper-bearing carbon steel and a carbon steel of special composition. The latter is used only in the architectural treatment of the tower bracing above the roadway; the copper-bearing carbon steel, principally in the end panels of the floor structure and in the railing. Silicon steel occurs only in the towers and the suspended structure while the two remaining grades may be found in all divisions of the bridge.

MANUFACTURE: The four forms in which practically all of the rolled steel was used for this structure are plate, shape, flat and rivet-rod. The term "flat" as used in this report means any plate less than 10 in. in width. Some of the sections, such as plates of several inches thickness and round bars of large diameter, are referred to as "thick material." Each of the four forms, with minor exceptions, was made in a different plant, though all steel was made in basic open-hearth furnaces. The operations involved in producing each of these rolled steels were essentially the same.



The process employed in making the above products followed standard practice. The operations, listed in the order in which they are conducted, include melting of raw materials; pouring the metal into ingot molds; stripping these molds from the ingots and placing the latter in soaking pits or furnaces to prepare them for preliminary (or, in some cases, final) rolling; rolling the ingots on a blooming mill and then shearing the resultant section to semi-finished pieces called slabs, blooms or billets, depending on their size and shape; reheating these pieces in furnaces and rolling them on a finishing mill to the desired form; and, finally, shearing them to yield the finished mill product. Movement of material in ingot form from open hearth furnace up to the blooming mill (which produced semi-finished material, as slabs for plates, blooms for shapes, or billets for smaller forms) was the same for all products. Beyond the blooming mill the semi-finished material passed through the rolling process which was suited to produce the desired form and which was consistent with that form except in the case of plates. Slabs for the latter were rolled in two ways. Those which produced plates wider than 60 in. were rolled on a sheared-plate mill, that is, the plate was rolled in such fashion that it had to be sheared to size on all four edges. Those slabs which produced narrower plates were rolled on a universal-plate mill, the finished plate having rolled edges but sheared ends.

IDENTIFICATION: Identification by melts is standard practice in the manufacture of metallic materials; the same scheme being followed for all products in the early stages of the process. Material is charged by melts in soaking pits and other reheating furnaces where it is located by means of charging sheets which record position of pieces in the furnace, or, as in continuous furnaces, by means of markers which serve as dividing lines between melts as well as parts of a melt. In other steps the means vary but identity of the melt is maintained. Since tests in this work were to be made on material from top and bottom cuts, the pieces rolled from these portions of ingots had to be followed through the operations.

The identity of the structural steel was carried out not only to permit selection of specimens but also to insure the use of correct grades in fabricated members composed of more than one grade.

The only material that did not require some means of identification between ingot and finished product was the I-beam material. The ingot passed directly from blooming mill to finishing mill. The product was, of course, identified by reference to the soaking-pit charging records.

To preserve the identity of slabs, from which the plates are rolled, they were stamped, while quite hot, at the blooming mill during shearing operations. The steel stamping included the number of the melt and a letter which indicated the slab's location in the ingot. The complete marking was so placed that the top end of the top cut or the bottom end of the bottom cut could be identified. This end was recorded on the charging sheet when the slab was placed in the reheating furnace preliminary to being

the center of the main span the
les are only ten feet above the
dway. Note the steel curbs and
handrails

rolled into a plate. In such a system of identification, however, identity of the ends may be lost in the course of drawing the slabs out of the furnace and placing them in position for rolling. Furthermore, identity of the ends may also be lost, as in the case of sheared plates, because of peculiarities in rolling procedure. In the sheared-plate mill the slab usually undergoes both transverse and longitudinal rolling. In the event the final rolling operation is performed transversely then what were originally ends of the slab finally become the sides of the plate. Such confusion in respect to ends cannot occur in connection with universal-plate rolling because the width of the slab is made such that the slab passes through the mill only in the direction of its length which is always measured lengthwise of the ingot. Identity of the ends as it affects testing of plates will be referred to subsequently.

Blooms, from which were rolled all angles larger than 4 by 4 as well as other shapes above the same size, were not stamped for identification because they moved directly from blooming mill through a continuous furnace to finishing mill. In the continuous furnace, markers were used to indicate divisions between melts and cuts.

Billets, (which produced rivet-rods, flats and small shapes) were seldom marked with respect to location in the ingot because orders on the mill usually required these forms in small quantities, calling for only a part of a melt or a part of an ingot. Those rolled into rivet-rods and small shapes were identified only as to melt. Billets for flats also were stamped only with the melt number except in the few cases when sizable orders were placed with the mill. In such instances location in the ingot was included with the markings.

The finished material was stamped with the number of the melt from which it had been produced. Rivet rods were identified by means of stamped metal tags attached to bundles of these pieces. Every individual piece of other products was steel-stamped with this number. To facilitate identification at the fabricating shops, the ends of silicon steel products were painted with green paint and of copper-bearing steel with yellow paint.

All specimen material was marked with melt number, and also with cut notation in cases where identity had been extended that far. The latter occurred in all specimen material excepting that which represented rivet-rods, small shapes and most of the flats.

CHEMICAL AND PHYSICAL REQUIREMENTS: Chemical and physical properties specified for each of the three grades of rolled steel were, in part, those given in Table 3. These requirements, however, had some modifications that have not been indicated in the table.

The restrictions in connection with chemical properties pertained to the content of different elements. These reservations limited the amount of manganese in silicon steel to 1.20% as a maximum; fixed a range of 0.05% to 0.15% for the amount of silicon in structural carbon steel; permitted an over-run of 10% in carbon, 25% in phosphorus and 25% in sulfur on check analysis; and established 0.20% as the minimum amount

TABLE 3.—SPECIFIED CHEMICAL AND PHYSICAL PROPERTIES FOR STRUCTURAL STEELS

	Carbon Steel (a)		Silicon Steel (b)
	Structural	Rivet	
Carbon (max.).....	-----	-----	0.40
Phosphorus (max.):			
Acid Process.....	0.06	0.04	0.06
Basic Process.....	0.04	0.04	0.04
Sulfur (max.).....	0.05	0.045	0.05
Silicon.....	-----	-----	0.20 to 0.45
Manganese } See text	-----	-----	-----
Copper }	-----	-----	-----
Tensile strength, lbs. per sq. in.....	60,000 to	52,000 to	80,000 to
	70,000	60,000	95,000
Yield Point, Minimum, lbs. per sq. in.....	36,000	30,000	45,000
Elongation in 8" minimum, per cent*.....	1,500,000	1,500,000	1,500,000
	Ten. Str.	Ten. Str.	Ten. Str.
Reduction of Area, minimum, per cent*	42	52	30

(a) Each group of ten melts of carbon steel to be used in the towers which consecutively have met the individual minimum yield point requirement had to have a minimum average yield point of 38,000 lbs. per sq. in.

Tensile tests of specimens from thick material or that of a thickness or diameter greater than $1\frac{1}{4}$ inches had to show ultimate strength and yield point as great as the minimum specified, a minimum elongation in two inches of 1,600,000 tensile strength, and a reduction in area of not less than 30 per cent.

(b) Each group of ten melts of silicon steel to be used in the towers which consecutively have met the individual minimum yield point requirement had to have a minimum average yield point of 47,000 lbs. per sq. in.

*See Text.

of copper that had to be present in steels specified to contain this element. The latter constituent averaged about 0.25% in amount. In the case of the material for the tower bracing enclosure plates, the carbon content was limited to 0.15% as a maximum on ladle analysis but no physical tests were prescribed.

Modifications in regard to physical properties defined requirements for the percentage of elongation and reduction of area in structural carbon and silicon steel. To arrive at the minimum percentage of elongation for structural carbon material between $\frac{3}{4}$ in. and $1\frac{1}{4}$ in. in thickness, one from the per cent elongation derived by the ratio given was to be deducted for each increase in thickness of $\frac{1}{8}$ in. (or fraction thereof) but the elongation in no case could be less than 18%. For the silicon steel over 1 in. thick the same amount of deduction was to be made, but for each increase of $\frac{1}{4}$ in. (or fraction thereof) and to a minimum of 14%. The percentage in reduction of area for the two grades was defined similarly. For structural carbon steel between $\frac{3}{4}$ in. and $1\frac{1}{4}$ in. in thickness, one from percentage reduction for each increase in thickness of $\frac{1}{8}$ in. (or fraction thereof) was to be made but the minimum could not be less than 35%. For silicon steel over $\frac{3}{4}$ in. thick the minimum was to be calculated the same way but the ultimate minimum could not be below 24%.

For ascertaining both kinds of properties, specifications required four groups of tests to be made but did not in all cases define requirements as to number of specimens nor their location. These tests comprised ladle and check analyses for composition, and mill and check tests for physical properties. In connection with chemical tests, the usual ladle or melt test had to be made on samples selected in the customary way, but with respect to check analyses, specifications did not prescribe the number nor location of samples. For gauging physical properties, the specifications required, in the matter of check tests, only a number deemed necessary to check the yield point, but in the case of the mill tests they were more definite. At least two tension and one bend test had to be made from each melt or variety of product such as plates, shapes or flats. In connection with this work, sheared and universal-mill plates, which are termed "plates," have been considered as different products. Specifications further required that in the event a variety in a melt differed $\frac{3}{8}$ in. or more in thickness, one tension and one bend test had to be made from both the thickest and thinnest material rolled. Every specimen for physical test had to have its longitudinal axis in the direction of rolling.

SELECTION OF SPECIMENS: Inasmuch as it was desired to determine the probable limits of chemical and physical properties, and because such limits would most likely be revealed by tests of material from top and bottom cuts of ingots, a procedure was established for the selection of physical and chemical test material from these parts of the melt. The specimen material was required to be taken from the top end of top-cut material and from the bottom end of pieces rolled from bottom cuts, but the cuts did not have to be from the same ingot to represent a given melt. In cases where extreme cuts of ingots were not applied to this work, or where one kind of cut only could be tested because the number of tests required per melt by specifications had to be used up in covering the prescribed gauge-variation in the product, this general method of specimen-selection had to be varied. It had to be altered also in connection with plate tests because after the rolling of plates had gotten under way it was felt that the identity of the ends of some slabs was being lost during reheating and rolling operations. In order that the top-most and bottom-most ends of these cuts would be tested, specimens were taken from both ends of plates.

The location of the physical test specimen in the rolled piece depended upon the shape of the product, but for all products, excepting sheared plates, the axis of the specimen was always taken lengthwise of the ingot or in the direction of all rolling. For testing angles, specimens were cut about $\frac{1}{2}$ in. in from the toe; for beams and channels, from a point in the web midway between the flange and axis of section; but for rivet-rods and most of the flats the full section as rolled was used to provide the specimen. For plates the axis of specimen coincided with the axis of piece as rolled, but in the case of sheared plates, it did not necessarily coincide with the axis of the ingot because the final rolling could be conducted transversely.

The samples for check analyses were drilled from the physical test specimen. In case

TABLE 4.—AVERAGE CHEMICAL AND PHYSICAL PROPERTIES OF THE STRUCTURAL SILICON STEEL IN MAIN STEEL
STRUCTURE—DETERMINED FROM LADLE ANALYSES AND MILL TESTS

	Towers				Suspended Structure				Grand Average
	Plates	Shapes	Flats	Average	Plates	Shapes	Flats	Average	
Carbon.....	0.30	0.29	0.30	0.30	0.30	0.31	0.35	0.31	0.30
Manganese.....	1.06	1.02	1.04	1.05	1.03	0.93	0.81	0.96	1.02
Phosphorus.....	0.018	0.024	0.021	0.020	0.014	0.22	0.019	0.018	0.019
Sulfur.....	0.028	0.035	0.031	0.030	0.029	0.037	0.030	0.033	0.031
Silicon.....	0.25	0.25	0.25	0.25	0.23	0.26	0.25	0.25	0.25
Ten. Str., lbs. per sq. in.....	86,900	87,000	87,000	86,900	86,400	87,000	90,300	86,900	86,900
Yield Point, lbs. per sq. in.....	50,500	57,800	53,600	52,400	52,900	58,900	54,200	55,100	53,300
Elongation in 8", per cent.....	21.2	23.6	22.0	21.8	21.8	24.3	21.3	22.6	22.1
Reduction of Area, per cent.....	45.4	52.4	50.2	47.5	47.4	50.6	49.5	48.6	47.9
Number of Melts.....	137	67	48	252	57	66	11	134	386
Number of Tests.....	534	163	114	811	280	154	28	462	1,273

TABLE 5.—AVERAGE CHEMICAL AND PHYSICAL PROPERTIES OF THE STRUCTURAL CARBON STEEL IN TOWERS AND
SUSPENDED STRUCTURE—DETERMINED FROM LADLE ANALYSES AND MILL TESTS

	Towers				Suspended Structure				Grand Average
	Plates	Shapes	Flats	Average	Plates	Shapes	Flats	Average	
Carbon.....	0.19	0.19	0.22	0.20	0.17	0.15	0.21	0.17	0.19
Manganese.....	0.56	0.60	0.50	0.56	0.58	0.60	0.49	0.57	0.56
Phosphorus.....	0.016	0.016	0.015	0.016	0.012	0.017	0.017	0.017	0.016
Sulfur.....	0.028	0.036	0.033	0.032	0.030	0.038	0.035	0.036	0.033
Silicon.....	0.09	-----	-----	-----	0.12	0.07	0.09	0.08	-----
Ten. Str., lbs. per sq. in.....	64,700	65,000	64,900	64,900	65,700	65,200	65,800	65,300	65,100
Yield Point, lbs. per sq. in.....	39,500	42,600	41,100	40,800	41,500	44,500	42,100	43,300	41,400
Elongation in 8", per cent.....	27.1	29.0	27.9	27.8	27.6	28.4	28.4	28.2	27.9
Reduction of Area, per cent.....	51.0	56.4	54.0	53.3	55.9	57.5	56.4	56.9	54.6
Number of Melts.....	226	187	152	565	45	203	98	347	912
Number of Tests.....	755	437	342	1,534	202	524	223	949	2,483

of plates, when the procedure for selecting specimens was changed, the drillings were taken from the top specimen showing the highest tensile strength and from the bottom one which gave the lowest strength. Such selection of samples had for its basis the assumption that physical properties are controlled principally by chemical composition.

During the course of the work, occasion arose for testing the entire product of a silicon steel melt which contained 21 ingots. These ingots, all of the same size, were first cut into either three or four slabs with differences in weight allowed to produce the desired lengths of plates and then rolled into 20 by $\frac{1}{2}$ in. universal-mill plates. The resultant product was tested physically at both ends and found to vary in tensile strength from 93,600 to 78,200 lb. per sq. in., while two check analyses showed 0.35% carbon for the top cut and 0.27% for the bottom.

The various chemical and physical properties of structural, silicon and carbon steels in the towers and the suspension structure are shown in Tables 4 and 5.

HEAT-TREATED EYE-BARS

The eye-bars incorporated in the structure are located in the cable anchorages and there serve to fix the ends of the cables. The bars spread fanwise in chains of three-link lengths from the end of a cable to girders embedded near the bottom of the anchorage. The eye-bars are connected between themselves, to the strand shoes around which cable wires are looped, and to the girders, by means of pins. There are 1464 eye-bars in the anchorages and they weigh 2520 tons.

In dimensions these eye-bars are alike only in respect to width. This is 10 in. They vary in thickness from $1\frac{1}{4}$ in. to $2\frac{1}{8}$ in., and in length, or between centers of pin-holes, from 33 ft. 4 in. to 66 ft. $8\frac{1}{2}$ in. The pin-holes have a diameter of either $10\frac{1}{32}$ in. or $11\frac{17}{32}$ in. for 10 in. and $11\frac{1}{2}$ in. pins, respectively. The bars are made of quenched and drawn carbon steel which originated in basic open-hearth furnaces.

For ascertaining quality of the material, the methods of analyses differed somewhat from those outlined under structural steel. Unlike the means employed in the case of the latter, eye-bars had to be tested full size in order to obtain the desired information with respect to their physical properties. Such a testing procedure demanded some kind of system for grouping bars into lots which could be represented by test bars and for this reason the history of each bar became a matter of prime importance.

MANUFACTURE: The manufacturing process proved to be uniform with respect to principal operations. Briefly, the procedure consisted of rolling slabs through a universal-plate mill into flats of the desired section and length, forging the heads and, finally, heat-treating the bars. In the last step, members were heated and quenched one at a time, but drawn two at a time.

IDENTIFICATION: Each eye-bar received numbers, by which it was known thereafter, at the rolling mill. The identifying marks consisted of melt number and slab number. They were steel stamped on the flat deep enough and at such place that

subsequent operations of forging and heat-treatment would not eradicate them. Since the slab number indicates location in the ingot, the eye-bar, after it had been completely manufactured, could be traced to a top, bottom or intermediate portion of an ingot in a particular melt.

CHEMICAL AND PHYSICAL REQUIREMENTS: Specifications set limits for amount of phosphorus and of sulfur in the material and for physical properties of heat-treated members. These requirements are given in Tables 6 and 7, respectively.

For obtaining their physical properties, specifications required full-sized and also Brinell hardness tests to be made on the completed members. At least 3 bars for each 100 needed for the work had to be tested to destruction. All bars, on the other hand, had to be subjected to Brinell hardness tests as set forth in the specifications of the American Society for Testing Materials entitled "Standard Methods of Brinell Hardness Testing of Metallic Materials (Designation E10-27)". The results of the latter tests were to be considered merely as a rough measure of the strength of individual bars.

RESULTS OF TESTS: The two prescribed kinds of physical tests were used in conjunction with each other for determining acceptability of the undestroyed eye-bars. The means adopted for ascertaining their strength had as its working basis the relation between the strengths of bars destroyed in test and the hardness numbers obtained from the same bars by Brinell test. By comparison of these hardness numbers with those obtained by Brinell test from the undestroyed bars, acceptability of the latter was determined. Test bars, however, were picked not alone for establishing the above relation but also with a view of investigating the effects on their strength, of irregularities in manufacture.

For the purpose of selecting test specimens that would be representative of lots into which the bars had been divided in a general way as manufacture progressed, a rather complete history of each bar was available. Before heat-treatment, the bar's chemical analysis was known, its location in the ingot and its physical properties. After heat-treatment, additional information in the form of temperature records, Brinell hardness results and notations about mechanical treatment were at hand. With these data as a guide, bars were selected and tested during progress of the work for the following principal reasons, aside from those concerning hardness results: Wide variations in chemical properties as compared to the general run of bars; unusual heat-treating temperature records; wide variations in the preliminary or mill tests which the eye-bar manufacturer required on untreated material; reforging and retreatment operations and excessive cold-working occasioned by straightening operations.

The average results of all full-sized tests, together with the extremes, are given in Table 7. These results have been arranged with respect to location of test bar in the ingot. They have not been corrected, however, for the slight error found in them by means of calibration tests conducted on the testing machine used for the work and the Emery testing machine at the National Bureau of Standards, U. S. Department of

TABLE 6.—AVERAGE* CHEMICAL PROPERTIES OF THE HEAT-TREATED EYE-BARS—
DETERMINED FROM LADLE AND CHECK ANALYSES

Test**	C	MN	P	S	Si
Specified (max.):					
Ladle.....	---	---	0.04	0.05	---
Check.....	---	---	0.05	0.063	---
Ladle:					
Maximum.....	0.39	0.74	0.037	0.048	0.14
Average.....	0.35	0.64	0.024	0.036	0.09
Minimum.....	0.30	0.55	0.012	0.012	0.06
Top:					
Quarter-Point:					
Maximum.....	0.45	0.78	0.042	0.056	0.18
Average.....	0.38	0.64	0.025	0.039	0.10
Minimum.....	0.26	0.53	0.011	0.025	0.17
Center:					
Maximum.....	0.49	0.77	0.043	0.054	0.18
Average.....	0.39	0.65	0.025	0.038	0.11
Minimum.....	0.26	0.53	0.011	0.023	0.07
Bottom:					
Quarter-Point:					
Maximum.....	0.44	0.76	0.037	0.046	0.17
Average.....	0.32	0.62	0.021	0.033	0.10
Minimum.....	0.26	0.50	0.010	0.024	0.07
Center:					
Maximum.....	0.46	0.74	0.036	0.046	0.18
Average.....	0.32	0.63	0.021	0.032	0.10
Minimum.....	0.26	0.52	0.010	0.023	0.07

*63 Melts.

**See text for location of samples for check analyses.



Commerce. The results of these tests on a 10 by 2 in. by 16 ft. 8½ in. bar indicated that for the same elongation the eye-bar testing machine loads were slightly higher than those of the other machine.

The Brinell hardness test was applied to the member at points midway between its edges. The test bars had both sides measured for hardness at 2-ft. intervals. Of those bars not tested to destruction the short ones had one side Brinell tested at three places, or near each head and at mid-length; the long bars, at other points in addition. The highest average hardness number of the bars incorporated in the structure proved to be 217, the lowest, 179. Average hardness numbers of the test bars are recorded in Table 7.

The average chemical properties of the eye-bars are given in Table 6. For the check analyses shown there, samples were taken from at least one top and one bottom bar in each melt. The test material was obtained from the center of the flat's cross-section with a ¾-in. diameter drill. That a comparison of compositions between this point and some average place could be made, drillings were also taken from the quarter-point in the same section or from a point located half way between center and edge.

TABLE 7.—AVERAGE PHYSICAL PROPERTIES OF THE HEAT-TREATED EYE-BARS—
DETERMINED FROM THE FULL-SIZED TESTS

Location of Bar in Ingot	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 18', per cent	Red. of Area, per cent	Brinell No.*	No. of Tests
Specified:						
Minimum.....	80,000**	50,000**	8.0	-----	-----	-----
Top:						
Maximum.....	98,100	63,200	11.8	53.7	214	-----
Average.....	92,400	59,400	10.0	40.5	203	20
Minimum.....	87,100	56,200	8.8	24.8	186	-----
Intermediate:						
Maximum.....	91,400	58,600	11.7	45.0	211	-----
Average.....	89,400	57,400	10.6	42.1	197	5
Minimum.....	87,900	55,800	9.4	35.5	187	-----
Bottom:						
Maximum.....	91,200	58,100	13.4	50.5	209	-----
Average.....	85,500	54,700	11.2	44.4	194	19
Minimum.....	79,700	50,400	7.9	37.3	179	-----
Grand Average (a)	89,100	57,100	10.6	42.4	198	44 (b)

*Hardness number of any bar is an average result.

**Values are minimum for any single test. Any twelve consecutive tests had to have an average ultimate strength of 85,000 and yield point of 53,300 lbs. per sq. in.

(a) 39 Melts.

(b) 1 bar, 10"x1¼"; 28 bars, 10"x2"; 15 bars, 10"x2⅛".

WIRES AND ROPES

The parts of the bridge composed of wires, all of which have been galvanized, are cables and their appurtenances. The latter are suspender ropes, hold-down ropes and wrapping wire. The cables, made up of 0.192 in.* diameter wire spliced with sleeves, extend from anchorage to anchorage and pass over the towers. Suspender ropes are looped over bands attached to the cables and serve as hangers for the floor system. These particular ropes have a nominal diameter of 2 11/16. in. Hold-down ropes which are of like form but 2 7/8 in. in diameter are attached to the cables in a similar manner but for the purpose of holding down the cables at the shore-ends of the side spans. The two kinds of ropes and the cables constitute integral parts of the bridge. The hand ropes and the wrapping wire are accessories. Hand ropes, 1 in. in diameter, are attached to the cables by means of vertical rod supports. Wrapping wire 0.148 in. in diameter, is used to cover portions of the cables not otherwise sealed, or, generally speaking, those parts between cable bands.

Wires used in the bridge amount to 23,185 tons. Of this quantity 21,597 tons entered into formation of the cables, 1211 tons into construction of suspender and hold-down ropes while the remainder went into hand ropes and wrapping wire. This material, with the exception of some cable wire and the ropes, was shipped to the bridge site in the form of bundles or coils after it had been dipped in a wax (known as Socony-Vacuum E. F. 205-15) and tied with 1/8-in. tarred marline. A very small portion of the waxed cable wire was spliced and reeled before being sent out but the ropes were manufactured completely, oiled and put on reels for shipment.

For the purpose of reviewing the quality of these materials, this portion of the report has been divided into three sections: Cable Wire; Ropes; and Wrapping Wire.

CABLE WIRE: Wire manufacture is an intricate process; at least it embraces more practices than are used for making any other kind of bridge material. When this is borne in mind, as well as the fact that the huge tonnage of cable wire required for the structure meant production of thousands of coils, the necessity for a system of identification becomes apparent as an important requirement for ascertaining the quality of each melt. Accordingly, there follows a brief description of the process employed in producing the wire and the means of preserving its identity.

Manufacture: The manufacturing process, in its simplest steps, consisted of melting, rolling, patenting, drawing and galvanizing operations.

The steel, made in acid open-hearth furnaces, was produced in melts of small quantity and handled throughout the work by melts. The average melt contained 28 ingots, or 168 billets, which gave 154 coils of galvanized wire, each about 380 pounds in weight. For the cables, 735 acceptable melts were made to produce 113,341 coils of galvanized wire.

*Wire diameters are nominal and before galvanizing, when not otherwise stated.

The rolling procedure transformed ingots into rods and involved two operations. In the first one the ingot was brought to rolling temperature in a soaking pit, reduced in a blooming mill and then passed directly, without re-heating, through a continuous billet mill. In the second operation the billet was re-heated and rolled through a continuous mill into rod form.

The subsequent steps of manufacture, or patenting, drawing and galvanizing procedures, transformed the rod into the finished wire product. In the patenting process the rods underwent heat-treatment. The work was of a continuous type because as one coil passed through the furnace another was attached to the disappearing end, the joint being disconnected on completion of its journey. The treated coils of rods were then passed through other departments to be cleaned, pickled, coated with lime and baked, or, in other words, were prepared for the drawing operation. In the latter step the rods were drawn into wire by pulling them through dies progressively smaller in diameter. In the galvanizing operation, the coils of wire were passed through a molten-lead bath, acid and fluxing solutions, and finally, through a spelter bath to give it the zinc coating. The procedure was a continuous one, each coil being attached to the preceding one and then disconnected after it had been led onto a reel. The coils were removed from these blocks and then tested.

Identification: The system of identification described here was followed not only on wire for the cables but also on rope and wrapping wires. It was established for the purpose of preserving identity of the materials from ingot to finished product.

The means of identification employed varied as the material passed from one stage of manufacture to another because of conditions attending different steps. When not charged into furnaces immediately after stripping the molds from them, the ingots of a melt were stacked and the melt number painted on a few pieces in each pile. Through the soaking pit identity of the ingot was preserved in the usual manner, or by means of charging records. The individual billets, on the other hand, were stamped with the melt number at the billet mill because they not only had to be stored until rolled into rods but also had to be handled with other steels on beds provided for surface inspection. Between the billet and the galvanized-coil forms the lot system was employed, each lot being composed of material from the same melt and marked with tags. Finally, to each of the galvanized coils was attached a metal seal bearing a serial number by which the coil could be traced to its melt.

The only parts of the material that required identification as to location in the melt were the short pieces of billets from which samples were drilled for check analyses. These pieces were cut from the top end of top billets and the bottom end of bottom billets. Their identity was carried along from the ingot by closely following the ingot through the billet mill. As the top end of the ingot or the front end of the first billet-length arrived at the shears a short piece was cut off and marked with its identity; the bottom-end piece was burned off immediately after the last billet-length had been sheared.

Chemical and Physical Requirements: Specifications required chemical analyses, physical tests and the Preece test to be made on the material. Chemical analyses consisted of ladle and the check. Physical tests comprised tension and wrap tests. The Preece test was a test for gauging thickness of zinc coating.

For chemical requirements, specifications set limits for carbon, phosphorus and sulfur. On ladle analysis, the percentage of carbon could not exceed 0.85; phosphorus, 0.04; and sulfur, 0.04. On check analysis, which was to be made on any portion of the steel suspected of being segregated, the carbon could not be more than 10% above the limit set for it on ladle test; and in the case of phosphorus and sulfur, not more than 25% above this limit.

With respect to physical properties, specifications required tests to be made on both bright and galvanized wire. These tests consisted of tension tests (for strength, yield point and elongation) and bend or wrap tests. For the bright wire, no tension tests had to be made but it had to be capable of coiling cold without sign of fracture around a rod $1\frac{1}{2}$ times its own diameter. This test had to be made on 10% of the coils. For the galvanized wire, both the tension test and the wrap test were required but of these the wrap test and that for yield point had to be made on only 10% of the coils.

The Preece test and a wrap test were the means by which the quality of the zinc coating had to be determined. In the latter test, which was required on 10% of the coils, the wire had to be wrapped around a mandrel $1\frac{1}{2}$ in. in diameter, and undergo this operation without its coat flaking or showing cracks visible to the naked eye. In practice, however, this wrap test was rarely applied because the galvanizing withstood the more severe test of wrapping the wire around a rod $1\frac{1}{2}$ times its own diameter, or the test for determining ductility of the steel after galvanizing. For the copper sulphate or Preece test, four one-minute immersions were prescribed. This test had to be made on at least 5% of the coils in any lot of wire. If tests of any coils failed to meet the requirements, every coil in the lot had to be tested. During this investigation, if less than 20% of the coils failed to pass the test then only the failures were to be rejected, but if 20% or more failed, the entire lot had to be rejected.

Results of Tests: Of the various tests required on the wire, results of



TABLE 8.—AVERAGE CHEMICAL PROPERTIES OF THE WIRE STEEL—DETERMINED FROM
LADLE AND CHECK ANALYSES

Material	No. of Melts	Ladle					Check				
		C	Mn	P	S	Si	C	Mn	P	S	Si
Cable Wire	Specified*	0.85	---	0.04	0.04	---	0.935	---	0.05	0.05	---
	Max.	0.85	0.79	0.035	0.037	0.36	0.93	0.78	0.035	0.039	0.39
	Ave.	0.81	0.66	0.026	0.028	0.24	0.82	0.64	0.025	0.029	0.24
	Min.	0.78	0.54	0.020	0.023	0.15	0.73	0.51	0.014	0.020	0.14
Suspender Rope Wire	Specified*	0.85	---	0.04	0.04	---	0.935	---	0.05	0.05	---
	Max.	0.84	0.75	0.039	0.034	0.28	0.89	0.76	0.041	0.035	0.27
	Ave.	0.81	0.67	0.025	0.027	0.23	0.82	0.66	0.025	0.030	0.22
	Min.	0.78	0.57	0.020	0.024	0.18	0.76	0.56	0.019	0.024	0.16
Hold-Down Rope Wire	Specified*	0.85	---	0.04	0.04	---	0.935	---	0.05	0.05	---
	Max.	0.82	0.76	0.028	0.031	0.28	0.85	0.69	0.029	0.041	0.26
	Ave.	0.81	0.68	0.026	0.028	0.23	0.81	0.64	0.025	0.034	0.22
	Min.	0.79	0.61	0.023	0.025	0.18	0.78	0.56	0.021	0.027	0.17
Hand Rope Wire	Specified*	0.85	---	0.04	0.04	---	0.935	---	0.05	0.05	---
	Max.	0.83	0.72	0.033	0.031	0.28	0.88	0.69	0.034	0.031	0.27
	Ave.	0.80	0.68	0.028	0.029	0.23	0.83	0.68	0.026	0.027	0.23
	Min.	0.78	0.66	0.024	0.026	0.19	0.78	0.65	0.021	0.024	0.20
Wrapping Wire	Specified	0.50-0.55	---	0.04*	0.04*	---	0.50-0.55	---	0.05*	0.05*	---
	Max.	0.52	0.63	0.028	0.030	0.28	0.57	0.63	0.030	0.037	0.29
	Ave.	0.51	0.57	0.024	0.028	0.23	0.53	0.56	0.025	0.031	0.21
	Min.	0.49	0.54	0.020	0.027	0.20	0.50	0.53	0.020	0.025	0.16

* Maximum.

(1) 36 Melts analyzed for copper showed an average content of 0.07 of one per cent.

TABLE 9.—AVERAGE PHYSICAL PROPERTIES OF THE CABLE WIRE

	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.* (1)	Elong. in 10", per cent	Diameter, inches
Specified.....	220,000 min. (2)	160,000 min.	4.0 min.	0.192 (3)
Maximum (4).....	243,500	188,000	6.9	-----
Average**.....	235,600	182,600	6.3	0.1955 (5)
Minimum (4).....	225,300	176,000	5.7	-----

*Based on gross section which equals actual cross-section including galvanizing.

(1) Yield Point defined as the point where elongation of specimen between gauge points, originally 10 in. apart, is 0.70 of one per cent.

(2) Any 12 consecutive tests had to show a minimum average of 225,000 lbs. per square inch.

(3) Nominal ungalvanized diameter which was permitted a variation of plus or minus 0.003 in. Average bright wire diameter found to be 0.1916 in. from measurement of at least 20 coils per melt.

(4) Average of a melt.

**Results of 22,816 Yield Point tests (two per coil) representing the 735 melts.

(5) Galvanized diameter.

chemical and tension tests only are recorded in this report. The chemical properties of the wire steel are given in Table 8, the results of the tension tests made on the finished material, in Table 9.

Check-analyses results for the wire steel were derived from top and bottom samples of ingots. The samples were drilled out with either a $\frac{5}{8}$ -in. or $\frac{7}{8}$ -in. tool from the exact center in the cross-section of the short pieces of 2 by 2-in. billets, or on the billet's longitudinal center line, the region where segregation occurs. In Table 8, which covers not only cable wire but also rope and wrapping wire steel, two checks, one top and one bottom, were made on every melt.

For the cable wire results in Table 8, two methods of selecting samples were followed. In every tenth melt both samples were secured from the same ingot, while in the remainder, one sample was taken from the top of one ingot and one from the bottom of another.

In connection with the results in Table 9, which are based on the yield point tests, the condition of the specimen as cut from the coil had to be considered in determining test procedure. The specimen had an arc of about 30-in. radius from being coiled on a 5-ft. take-up block. Experiments disclosed the fact that, with a specimen 25 in. in length, the 12-in. to 14-in. central portion could be made straight, as measured with a straight-edge, by subjecting the specimen to a load of about 700 to 750 lb. in tension. Based on these data, and the desire to keep the initial or straightening load to a minimum in order that a maximum amount of the stretch defining the yield point might be measured, the length of the specimen was specified to be 25 in. and the initial load, 860 lb. This load was assumed to produce a stretch of 0.1% in a 10-in. gauge-length for wire diameters in the specified range, the stretch being based on a modulus of elasticity of 28,750,000 lb. per sq. in. which had been derived from tests.

The small portion of wire spliced at the wire plant and shipped to the bridge site

amounted to 2424 coils and was distributed over 243 reels. Specifications prescribed for the splice a minimum efficiency of 95% to be based on the average galvanized diameter of the wire and its specified minimum average tensile strength. The latter was 225,000 lb. per sq. in., the former, 0.1955 in., resulting in a minimum breaking load of 6420 lb. for the splice. Of the 2181 joints made at the mill, 214, or about one in every ten, were cut out for test. If a specimen failed to develop the specified load, operations were halted and the splice-making apparatus readjusted, but before resuming the work of splicing, each of three consecutive splices had to develop a strength of at least 6420 lb. The average breaking load in the regular tests proved to be 6920 lb. In the 5 tests which failed to meet the requirements, the lowest load was 5850 lb.

The sleeve, in this splice work, was pressed on the wire ends. In the operation, the wire ends, which had been threaded with pressure dies for a length of about $\frac{7}{8}$ in. and made slightly oval-shaped, were inserted into the sleeve, which had ends tapped for a length of about $\frac{3}{8}$ in., and the assembly then squeezed in a three-part die. As a consequence of this pressing action, the galvanized sleeve, which could not exceed $\frac{13}{32}$ in. in diameter according to specifications, increased in length from $2\frac{7}{16}$ in. to about $2\frac{1}{2}$ in. and decreased in diameter from about 0.41 in. to about 0.39 in.

ROPES: Three kinds of ropes are used in the structure, viz., suspender ropes, hold-down ropes and hand ropes. The suspender and hold-down ropes are somewhat unusual in design in that they have a strand center instead of an independent wire rope center. However, both kinds have a socket on each end. The hand ropes are 1-in. diameter strands socketed at both ends and composed of one center wire 0.208 in. in diameter and two layers totaling 18 wires of 0.196-in. diameter.

Manufacture: The material for making the wire, all produced in acid open-hearth furnaces, went through the same manufacturing process as did cable wire. Some exceptions occurred arising from the fact that wires of several different sizes were needed for making the ropes. The size of the rods varied as did also the number of drafts in reducing these sections to the desired wire diameter.

The work of manufacturing the ropes consisted of three principal steps, (1) forming, (2) pre-stressing, and (3) socketing. The manufacturer followed common practice in forming or constructing the rope but varied from it by using butt welds for splicing wires instead of brazing means. To produce the three different kinds of ropes in the required number of lengths, 29 lengths of suspender ropes were made, each about 7100 ft. long; one length of hold-down rope, 1870 ft. long; and 9 lengths of hand rope, each 3600 ft. long. These lengths are known as manufactured lengths and contained in most cases more than one member. For such lengths of ropes, splicing of wire coils became necessary and for the work the manufacturer used the butt-welding method. A comparison of the results from tests made on butt-welded and brazed joints gave a slight advantage to the work in the use of the former. In strength, the two kinds were practically equal, or about 50% of the wire strength. In uniformity of section

at the splice, however, the welded joint was considered superior because it had a continuity that is difficult to attain consistently in brazed joints.

The pre-stressing operations, required for the purpose of giving ropes the elastic properties desired, involved the two factors of time and load. For suspender ropes a load of 250,000 lb. was maintained for 14 hours as a general practice though in some cases the time period was extended to 16 hours. For hold-down ropes the load was 350,000 lb. during a stressing period of 12 hours. The hand ropes were held in tension for one-half hour under a load of 110,000 lb. For measuring the needed length of each piece, suspender ropes were subjected to a load of 92,000 lb.; hold-down ropes, 87,000 lb.; and hand ropes 36,500 lb.

The socketing operation was a simple one. After the ropes had been cut to length, the ends were spread into brushes and cleaned of zinc, oil and dirt. The ends so prepared were inserted into sockets and sealed in place with zinc.

Chemical and Physical Requirements: The requirements for composition and physical properties of individual wires were the same as those for cable wire. Specifications required physical tests on the suspender and hold-down ropes only. These tests had to be made for the purpose of determining the ultimate strength and the elastic behavior of each type. The strength tests consisted of two kinds; in one, called the double-part test, the specimen had to be stressed while looped over a sheave; in the other, or single-part test, the specimen had to be stressed in direct tension. From each manufactured length of rope one specimen for each kind of test had to be cut. If, after three or more double-part tests had been made, it was found that the ratio of the rope's strength in single-part to its strength in double-part was constant, the double-part tests could be omitted. In such a case the specified strength for a straight rope was to be considered as having the same ratio to the specified strength over a sheave as the ratio established by the preceding tests. Samples for both kinds of tests had to have attached to them, sockets from among the extra ones provided for use in the bridge.

Suspender ropes, in double-part tests, had to develop a total strength in both parts of not less than 1,100,000 lb.; and hold-down ropes 1,300,000 lb. The minimum efficiency of the former in double-part was found to be approximately 91 per cent. Based on this value, a strength of 606,000 lb. in single-part was necessary to develop the specified minimum double-part strength. The corresponding load in single-part for the hold-down ropes was not needed because no single-part tests were made for the purpose of gauging its strength in double-part.

Ropes tested straight, or in single-part, had to have strain measurements made in a gauge length of 100 in. to determine their elastic behavior. The stretch in 100 in. from an initial load of 5000 lb. to a load of 190,000 lb. could not exceed 0.5 in. for rope not previously stressed, nor 0.3 in. for rope having been subjected to repeated loading up to 190,000 lb., and the stretch of any rope could not vary more than 10% from the average stretch of all ropes tested. The initial load of 5000 lb. for the strain test was

increased to 20,000 lb. because the smaller load had proved insufficient, in the first tests, for straightening specimens of such large rope. This change accordingly led to a proportioned reduction of the maximum amount of stretch from 0.3 in. to 0.2757 in. for pre-stressed rope.

Results of Tests: The average chemical properties for wires used in all the ropes are given in Table 8, together with the requirements. The average physical properties are recorded in Table 10 for wires incorporated in suspenders. The results of double-part tests for suspender and hold-down ropes are shown in Table 11, while the results of single-part tests on suspender ropes are shown in Table 12, together with the requirements in each case.

In the various tables giving results of tests made on individual wires, the testing procedure was the same as that used in connection with cable wire except for the amount of initial load. This had to be varied to suit the nominal diameter of wires so that the load for the same modulus of elasticity found for cable wire would produce a stretch of 0.1% in a 10-in. gauge-length.

The double-part tests in Table 11 were made over a sheave having about the same diameter and elliptical shape as the cable band groove. The sheave conformed to the smallest circumferences or worst condition that would be found in service.

TABLE 10.—AVERAGE PHYSICAL PROPERTIES OF THE VARIOUS WIRES IN THE
2 11/16" DIAMETER SUSPENDER ROPES

Diameter* in inches			Tensile Test Results**			
Nominal	Bright	Galvanized	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 10", per cent	No. of Tests
0.203	0.2000	0.2038	232,500	178,800	6.4	14
0.179	0.1759	0.1802	243,500	183,800	6.3	94
0.154	0.1511	0.1549	250,500	189,900	6.3	230
0.153	0.1502	0.1542	252,200	190,100	6.2	64
0.136	0.1332	0.1369	232,500	182,300	6.4	570
0.131	0.1284	0.1320	237,500	185,100	6.4	324
0.117	0.1142	0.1174	249,500	179,600	6.4	288
0.114	0.1111	0.1147	234,800	170,400	6.7	76
0.103	0.1000	0.1037	246,000	178,100	6.5	70
0.098	0.0954	0.0985	255,800	188,100	6.5	48

*Average bright wire diameter based on measurement of at least 20 coils per melt; and galvanized diameter, on Yield Point tests or 10% of the coils.

**All averages based on Yield Point tests.

The single-part tests in Table 12 are listed in the order in which manufactured lengths were made. It will be noted that two tests for each of the first three lengths are given. The additional tests resulted from the fact that in all these pre-stressing operations each manufactured length had to be cut in half and each half then stressed separately. One test for each pre-stressed length was made at the beginning of the work to gauge the uniformity of operations.

All single-part tests of suspender ropes shown in Table 12 were made without any more stressing than that performed in the pre-stressing operation. However, the specimen of the test noted (b) in the table was subjected to a second stressing in the testing machine.

WRAPPING WIRE: The specifications called for a soft annealed wire with the chemical and physical properties indicated in Tables 8 and 13, respectively. No test for yield point was required but the physical tests for other properties had to be made as on other wires.

The manufacture of this material followed the steps generally to be found in connection with other wires. The steel was made in acid open-hearth furnaces although this kind of material is usually produced by the basic process. The rods had a smaller diameter and received a different number of drafts in drawing the wire because the finished diameter had to be 0.148 in. The rods were not patented but the bright wire was heat-treated before being put through the galvanizing unit.

TABLE 11.—RESULTS OF THE DOUBLE-PART TESTS MADE ON PRE-STRESSED
SUSPENDER AND HOLD-DOWN ROPES

Suspender Ropes 2 11/16" Diameter		Hold-Down Ropes 2 7/8" Diameter	
Manufactured Length, No.	Ultimate Strength, pounds	Manufactured Length, No.	Ultimate Strength, pounds
Specified Min.	1,100,000	Specified Min.	1,300,000
1	1,315,200	1	1,472,400
2	1,327,400		
3	1,338,800		
Average	1,327,100	-----	-----

TABLE 12.—RESULTS OF THE SINGLE-PART TESTS MADE ON 2 11/16" DIAMETER
PRE-STRESSED SUSPENDER ROPES

Manufactured Length, No.	Ultimate Strength	Total Elong. in 100", inches*
Specified	606,000 min.	0.2757 max.
1 (a)	718,600	0.2596
1	738,800	0.2602
2	656,200	0.2667
2 (a)	726,000	0.2635
3 (a)	658,200	0.2743
3	664,200	0.2723
4	700,200	0.2542
5	702,800	0.2558
6	723,400	0.2489
7	725,200	0.2653
8	730,200	0.2673
9	698,800	0.2630
10	706,800	0.2565
11	715,800	0.2601
12	673,400	0.2663
13	715,800	0.2662
14	705,000	0.2662
15	659,000	0.2855
16	686,800	0.2659
17	689,000	0.2822
18	690,600	0.2716
19	681,400	0.2742
20	679,600	0.2786
20	-----	0.2698 (b)
21	727,600	0.2540
22	694,200	0.2550
23	728,000	0.2524
24	725,400	0.2523
25	725,600	0.2572
26	722,600	0.2589
27	692,000	0.2576
28	710,600	0.2589
29	691,000	0.2559
Average	702,900	0.2638

*Between loads of 20,000 and 190,000 pounds.

(a) Specimen cut from manufactured length at point adjacent to double-part specimen of corresponding number in Table 11.

(b) Not included in average; see text.

TABLE 13.—AVERAGE PHYSICAL PROPERTIES OF THE WRAPPING WIRE

	Tensile Strength, lbs. per sq. in.*	Elong. in 10", per cent	Diameter, inches
Specified.....	85,000-100,000 (1)	10.0 min.	0.148 (2)
Maximum (3).....	93,700	15.4	-----
Average**.....	92,200	14.8	0.1515 (4)
Minimum (3).....	91,000	13.7	-----

*Based on gross section which equals actual cross-section including galvanizing.

(1) Any 12 consecutive tests had to show a minimum average of 90,000 lbs. per sq. inch.

(2) Nominal ungalvanized diameter which was permitted a variation of plus or minus 0.003 in. Average bright wire diameter found to be 0.148 in. from measurement of at least 20 coils per melt.

(3) Average of a melt.

**Results of 188 tests (one per coil) representing the 11 melts.

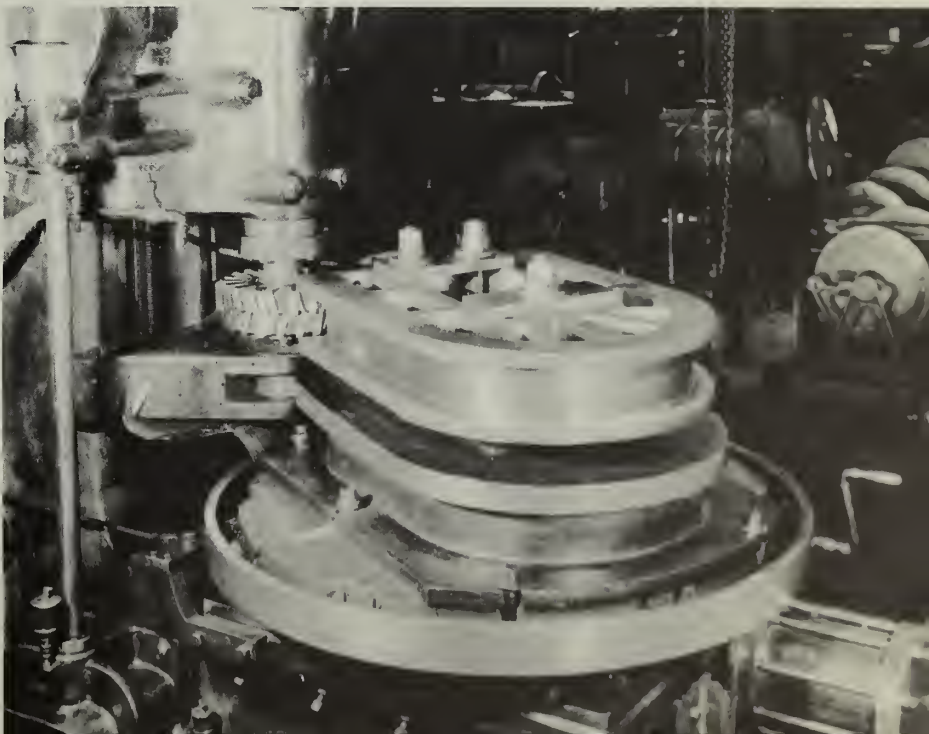
(4) Galvanized diameter.

CAST STEEL

Steel castings used in the structure serve a number of purposes. Saddles on top of the towers provide support for the cables; strand shoes are the means by which the cables are connected to the anchorages; while the cable bands serve, among other things as the method for attaching suspender and hold-down ropes to the cables. Cast steel is also employed at points in the towers and pylons where the suspended structure is hinged to them. Here it has been shaped into sleeves for the rocker links; into slotted castings for the lateral bracing system; and into expansion shoes for roadway members. Miscellaneously, cast steel is used for such members as pin nuts. All castings in the bridge total 1573 tons in weight of which 1545 tons, or nearly all of the material, consists of equipment for cables.

Obviously the utility of these castings necessitated members of various sizes and shapes. Each cable saddle is made of three segments bolted together to form a unit with extreme dimensions of 21 ft. 7 in. in length, 10 ft. 0 in. in width and 10 ft. 6 $\frac{5}{8}$ in. in height. It weighs about 150 tons. A complete band has a bore-diameter of 35 $\frac{15}{16}$ in. and a length ranging from 1 ft. $\frac{1}{2}$ in., for those used in attaching hand ropes to the cables, to 6 ft. 3 in. for the longest suspender rope bands. The smallest band weighs about one-half ton, and the largest about 4 tons. A strand shoe has extreme dimensions of 2 ft. by 3 ft. 1 $\frac{1}{2}$ in. by 7 $\frac{3}{8}$ in. and

Strand shoe after
being grooved for a
cable-strand





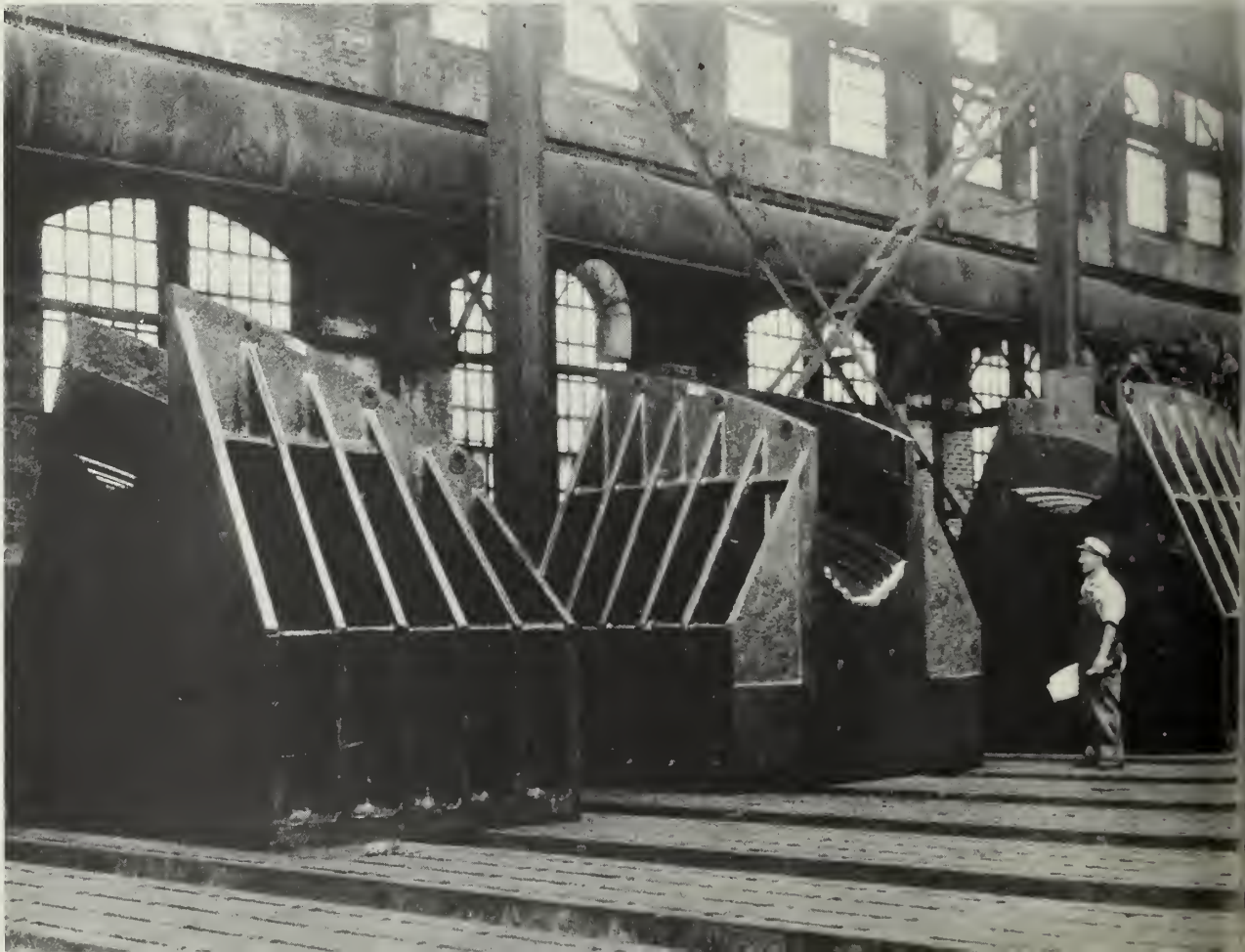
Cable-saddle segment before the heads and the runner at the left had been removed. Molten metal was poured into the mould through this runner

weighs about 820 pounds. These three types of members constitute the bulk of the cable fittings. None of the remaining castings is more complicated in shape.

Of all the castings, cable bands proved to be the hardest to produce. The difficulty encountered in their production pertained not to properties of the steel, nor to machine work but arose in connection with soundness of members. The defects that developed in the course of the work occurred not only in the cable bands, most of which had to be replaced before final acceptance, but also in the saddles, two segments of which had to be made over, and even in the pin nuts and rocker link sleeves, both types of which had to be replaced in their entirety.

Since the process employed in producing cable bands is the same as that for making other members, an outline of cable-band manufacture is given below. In addition, the machine work on the saddles is briefly described because it was a special process.

Segments of a cable-saddle ready for shipment



Manufacture: Seven manufacturers produced all castings for the bridge. Six supplied the cable bands, and the seventh made the other castings. Naturally, different methods of manufacture were followed on similar members, and in addition, different types of furnaces were used for melting the steel. Nevertheless, founding operations were conducted along customary lines.

In producing a steel casting, three main operations are employed. The first is molding; the next, casting, or pouring the metal; and the last, annealing. The mold is constructed of sand prepared in different ways and into it molten metal is poured. After the steel has solidified the resultant member is cleaned of sand and subjected to some form of annealing process. The method used in making the mold determines, to a great extent, the soundness of the casting; the other operations or making of steel and heat-treatment, control the chemical and physical properties of the metal.

Molding procedure varies in different plants and even in the same plant dissimilar methods are employed for different castings. It includes, among other things, provisions for introducing molten metal into the mold as well as means of supplying hot metal to the casting during the period of solidification. As already stated, the cable bands were made by several manufacturers, yet in the four plants where any appreciable number of the same type member were produced three distinctly different methods of molding and casting were employed. In Fig. 8 are three sketches which indicate the basic means employed in successfully feeding the cable band castings by the use of "heads." In all these schemes the metal was introduced into the mold at its lowest point, or in the standard way. However, in the case of Plan No. 3 the mold was turned upside down immediately after it had been filled with molten metal.

These molding systems were developed after much time and material had been lost in an attempt to produce acceptable castings. The two kinds of defects for elimination of which the manufacturer strove successfully consisted of cavities and cracks. Cavities occurred in the heavier sections of the castings, particularly in those parts which were at some distance from the head or source of hot metal. They were found generally in the region of the bolt housings. Cracks or tears were found on the bore side of the band and in the suspender-rope grooves. This type of de-

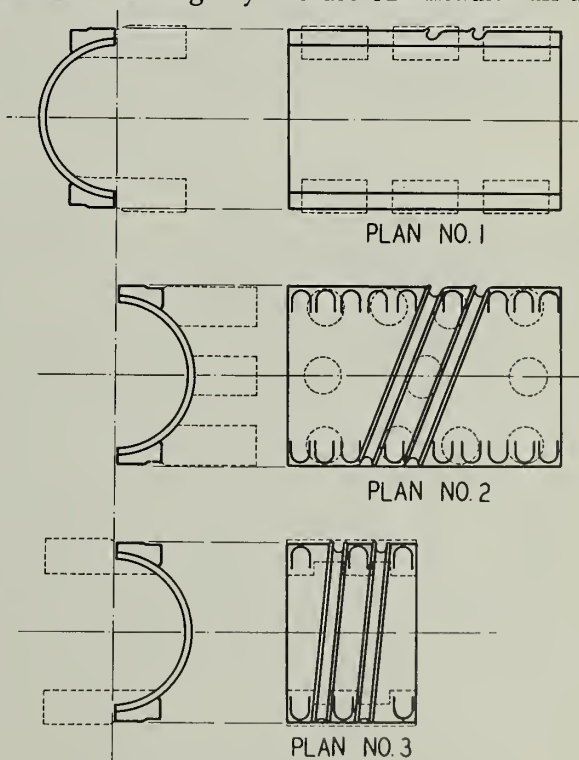
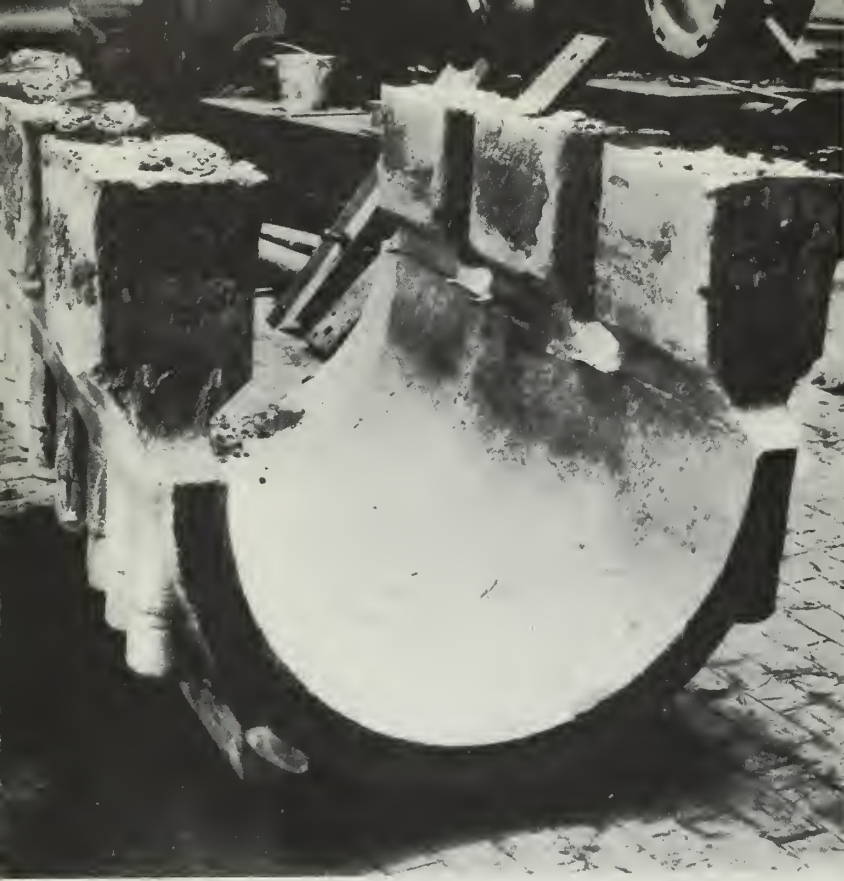


FIG. 8—Three methods of molding cable bands



Half cable-band before heads had been removed

fect was not readily apparent until the surface metal had been removed by machine work; nor were cavities in bolt housings and in parts of other castings, until opened by means of a drill. To eliminate waste of material through the machining method, and to insure solidity in all parts of the castings, various methods of moldings were tried in connection with the bands until sample castings, cut up into sections, proved the method to be successful. The method of producing satisfactory cable band castings should have been proved by destructive exploration of trial castings before attempting to manufacture for the order. For proving soundness of such members as pin nuts and rocker sleeves, sample ones were machined before pro-

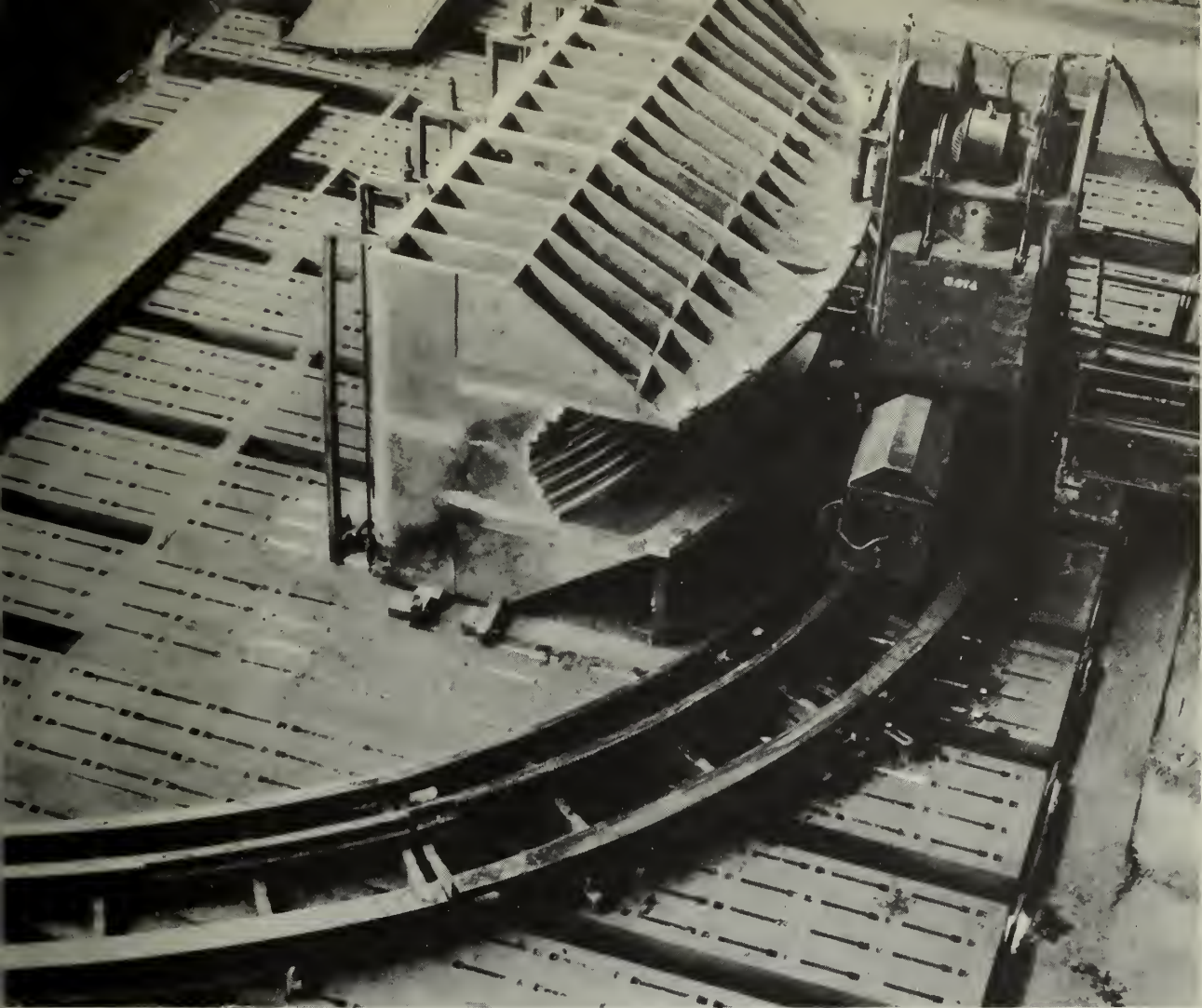
duction of the remainder of the replace castings got under way.

Chemical and Physical Requirements: The cast steel members had to be manufactured in accordance with the specifications for Class B steel castings of the American Society for Testing Materials (Designation A-27-24) with exceptions as noted in Table 14.

The required number of tests varied with the size of the castings. One tension and bend test had to be made for each melt and annealing charge or at least two such tests per melt where castings weighed under 500 pounds apiece. Each of the heavier castings had to be tested by means of one tension and one bend specimen. In the case of the heaviest castings (the saddle segments) each of which required two melts for its production, two tension tests and one bend test had to be made for each member.

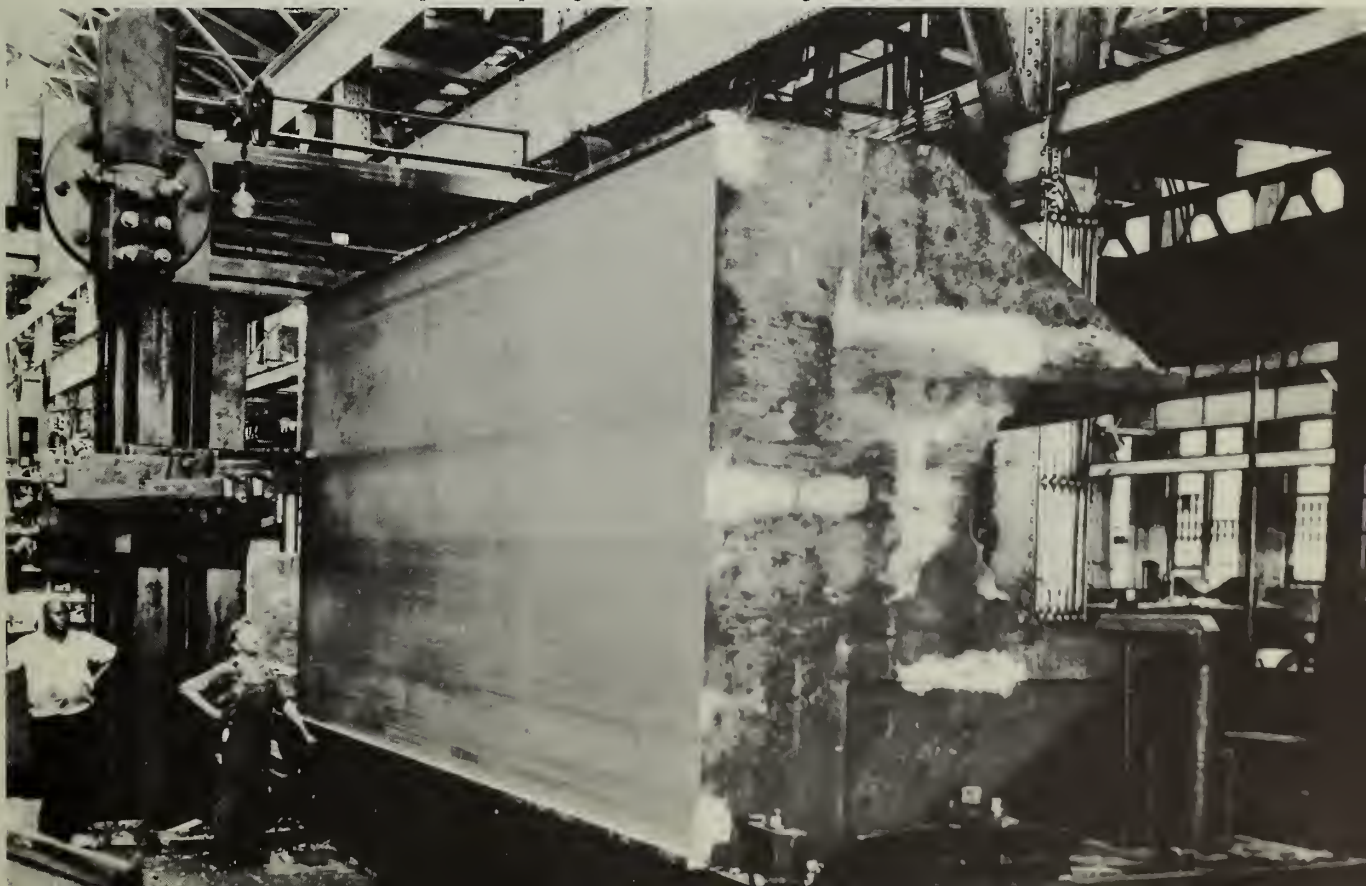
Results of Tests: The average chemical (ladle) and physical properties of each of the three kinds of steel of which the castings are made are listed together with specified requirements, in Table 14. The specimens used to determine physical properties were machined from coupons cast integrally with the castings. The coupon had cross-sectional dimensions of about $1\frac{5}{8}$ by 4 in. for all castings except the saddle segments. For the latter, the coupon's dimensions were made about $2\frac{1}{8}$ by 4 in.

Machine Work on Cable Saddles: The work of machining surfaces of the cable saddles took place in three steps. In the first, the ends and base of each segment were planed; in the second, the cable way was machined out after the three segments comprising a saddle had been bolted together; and in the last step, the base was planed to obtain an even, finished surface.



Above: Cable-saddle set up for machining cable-way

Below: Cable-saddle set up for final planing of base with all three segments assembled



The first and third steps were performed on planers. The second one required special apparatus to shape the cable trough and the grooves at its bottom. The arrangement for this work consisted of a milling machine mounted on a track which had been built to such a radius that the machine could travel along the track and at the same time cut out the individual grooves to the required radius.

TABLE 14.—AVERAGE CHEMICAL AND PHYSICAL PROPERTIES OF THE ANNEALED CAST STEEL—DETERMINED FROM LADLE ANALYSES AND FOUNDRY TESTS

	Specified	Process of Manufacture			Grand Average
		Acid Open Hearth	Acid Electric	Basic Open Hearth	
Carbon.....	-----	0.28	0.21	0.27	0.25
Manganese.....	-----	0.66	0.69	0.63	0.67
Phosphorus:					0.028
Acid.....	0.06*	0.035	0.023	-----	-----
Basic.....	0.05*	-----	-----	0.019	-----
Sulfur.....	0.05*	0.037	0.036	0.034	0.036
Silicon.....	-----	0.36	0.42	0.42	0.40
Ten. Str., lbs. per sq. in.	65,000**	72,700	69,600	70,900	71,100
Yield Point, lbs. per sq. in.	35,000**	38,400	41,400	37,900	39,500
Elongation in 2", per cent	20**	28.1	28.4	24.9	27.9
Reduction of Area, per cent.....	30**	42.4	41.5	37.3	41.6
No. of Melts.....	-----	166	140	54	360
No. of Tests.....	-----	509	381	96	986

*Maximum.

**Minimum.

FORGED STEEL

Steel forgings occur in various parts of the structure. They are used for rollers, pins, bolts, rope sockets and rope collars. The rollers are located on top of each tower between cap plate and cable saddle. The pins serve as connections between members in the anchorages, between ends of stiffening trusses and rocker links, and between rocker links and pylons or towers. Other pins, of relatively small diameter, occur at the pylons and towers or where the suspended structure is hinged with respect to these two points. The bolts join together the halves of cable bands and splay castings. A socket is attached to either end of suspender and hold-down ropes. The rope collars are riveted to top chords of the stiffening trusses at each point where the suspender rope leads down through the cover plate. The above members have been machined all over except in the case of the rope collars which had practically no finished surfaces and in the case of the bolts which had nearly all surfaces finished.

These members have been grouped here because their manufacture was essentially the same. The work performed on them may be divided into three main steps: forging, heat-treating and machining. In the first step the pins were forged from ingots into multiple-length pieces; the rollers and the sockets, from blooms into multiple lengths also; the suspender rope collars were drop-forged from flats; while in the case of the bolts, ends of rods were upset by forging methods to form heads of bolts. The heat-treatment to which each group of members was subjected fell under one or another of the three following methods, viz., quenching and drawing, annealing, and normalizing. The latter treatment was applied only to the nuts for the bolts.

For ascertaining the properties of all forgings excepting the suspender-rope collars, test specimens were selected with respect to location in the ingot.

Identification of cuts required no special work to be performed except in the case of bolts. The pins, rollers and sockets required little attention in the way of identification because, aside from heat-treating, there were no operations performed on them after forging from ingot or bloom before they were ready for testing. The bolts, on the other hand, required millwork and other operations to be performed on them and, therefore, had to be followed rather closely through the manufacturing procedure. These members only are treated below in some detail with respect to manufacture and identification. The chemical and physical requirements of pins, rollers and sockets are given in the accompanying table; the others are taken up in connection with requirements and results only.

The forged materials are treated under the following divisions: Cable Band Bolts; Cable Anchorage Pins; Stiffening Truss Rocker Link Pins; Tower Saddle Rollers; Suspender Rope Sockets; Allegheny Metal; and Suspender Rope Collars.

CABLE BAND BOLTS: Cable band bolts amount to 3978 in number. Some differences occur in their detailed dimensions but 3602 are alike. The remainder are similar except for their length. The larger lot of bolts has a $2\frac{3}{8}$ in. diameter and 2 ft. $5\frac{5}{8}$ in. length, the others a diameter of $2\frac{1}{2}$ in. but lengths varying from 2 ft. $7\frac{5}{8}$ in. to 3 ft. $2\frac{5}{8}$ in. under the head. The nut and head are hexagonal. Each bolt is finished all over except on the flats and top side of head and nut. These members are made of quenched and drawn carbon steel except for the nuts which are made of normalized carbon steel. All material is of basic open-hearth manufacture.

Manufacture: In addition to the main steps of forging and heat-treating mentioned before, the material for the bolts had to pass through rolling operations similar to those used in producing structural steels. In this procedure the ingot was made into blooms, the blooms into billets, and the latter into rods which were sheared into short lengths. On each of these short pieces a head was forged. These bolt-blanks were then heat-treated in a continuous furnace.

Identification: For the larger number of bolts the manufacturer used four melts, three of which furnished material for the bolts, and the other, for the nuts. Up to the

billet-mill stage this material was kept separate by melts like any structural steel, but beyond this point the manufacturer handled the top, bottom and intermediate cuts of each melt in groups and used a more definite means of marking the pieces which resulted from subsequent operations. The billets from each cut were hot-stamped with the melt number every 24 in. and, in addition, were painted on the ends to distinguish the cut. After they had been rolled into rods, the long melt numbers were replaced with single digits for intermediate cuts and with letters for other cuts. During the process of cutting the rods into short lengths the operator stamped each of these with the mark of the larger piece. From this point, the nut-blanks needed no further identification. The other small pieces were separated into groups having the same identifying mark and then headed with separate dies which had been engraved in order to raise the identifying number or letter on the head. These pieces were retained in groups for the heat-treating operation but after the blanks had been heat-treated they were piled, in the order treated, into lots of 51, except in the case of the top and the bottom cuts the number of which did not produce full lots. To each lot, which now contained blanks from the same melt and kind of cut, was assigned a number which was steel stamped on the head of each bolt. Each finished bolt then, by means of the markings on the head, could be identified with respect to melt, location in the ingot and heat-treatment lot.

Chemical and Physical Requirements: Specifications required a ladle analysis for composition and tests for physical properties. The requirements are given in Tables 15 and 16, respectively.

To determine physical properties, tension tests on both small and full-sized specimens had to be made. One tension and one bend test using standard 0.505-in. diameter specimens had to be made from each heat-treated lot of 50 bolts. Full-sized

TABLE 15.—AVERAGE* CHEMICAL PROPERTIES OF THE HEAT-TREATED CABLE BAND BOLTS—DETERMINED FROM LADLE AND CHECK ANALYSES

	C	Mn	P	S	Si
Specified (max.):					
Ladle.....	0.55	-----	0.04	0.05	-----
Check.....	0.605	-----	0.05	0.063	-----
Ladle.....	0.49	0.53	0.012	0.029	0.17
Check:					
Top:					
Quarter-Point.....	0.52	0.54	0.015	0.033	0.16
Center.....	0.56	0.56	0.015	0.037	0.17
Bottom:					
Quarter-Point.....	0.51	0.55	0.012	0.031	0.17
Center.....	0.50	0.55	0.013	0.031	0.17

* 3 Melts.

tests had to be made in sufficient number to provide a check on values determined from specimen tests.

Results of Tests: The average results of chemical analyses are given in Table 15 and of physical tests in Tables 16 and 17.

TABLE 16.—AVERAGE* PHYSICAL PROPERTIES OF THE HEAT-TREATED CABLE BAND BOLTS—DETERMINED FROM MILL TESTS**

	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 2", per cent	Red. of Area, per cent
Specified (min.)	95,000	60,000	21.0	45.0
Maximum	111,000	74,500	27.5	64.5
Average	101,300	69,300	24.6	62.4
Minimum	95,500	62,800	22.0	58.5

*80 Tests.

**Specimens taken midway between center and outside of bolt.

TABLE 17.—RESULTS OF THE FULL-SIZED TESTS MADE ON HEAT-TREATED CABLE BAND BOLTS

2 3/8" Diameter			2 1/2" Diameter		
Test No.	Tensile Strength, lbs. per sq. in.	Yield Point, lbs. per sq. in.	Test No.	Tensile Strength, lbs. per sq. in.	Yield Point, lbs. per sq. in.
Specified			Specified		
Min.*	95,000	60,000	Min.*	95,000	60,000
1 (T)	107,200	80,400	1	114,600	84,100
2 (B)	98,600	76,000	2	109,500	80,500
3 (T)	107,300	82,100	3	103,100	79,300
4 (B)	103,200	74,500	4	109,600	86,300
5	104,300	85,100	5	105,200	81,700
6	107,900	79,300	6	107,800	83,600
7	112,000	83,200			
8	114,400	84,800			
9 (T)	105,900	77,700			
10	114,000	63,400			
11	110,600	83,800			
12	111,000	83,200			
Average	108,300	79,500	Average	108,300	82,600

*Based on cross-sectional area at root of thread.

T—indicates top.

B—indicates bottom.

The chemical properties were determined from ladle and check analyses. For the latter, samples were taken by means of a $\frac{7}{8}$ -in. diameter drill at center and quarter points of the cross-section from two short pieces of the $5\frac{1}{4}$ in. square billets representing a top and a bottom cut of the melt.

The test results recorded in Table 16 were derived from the 0.505-in. specimen tests of the $2\frac{3}{8}$ -in. diameter bolts only because the $2\frac{1}{2}$ -in. diameter members were accepted on the basis of full-sized tests. Results of the latter appear in Table 17 together with those of the $2\frac{3}{8}$ -in. diameter bolts. With few exceptions the bolt or test-material was chosen at random from each lot. The exceptions occurred when lots contained some bolts treated in unusual manner. In this connection two special procedures were employed to straighten bolts and to fill out their heads: (1) cold-straightening and then strain-relieving by heat; (2) reforging followed by regular heat-treatment, cold-straightening and then strain-relieving.

CABLE ANCHORAGE PINS: These pins, of which there are 704 in both anchorages, are either 10 in. or $11\frac{1}{2}$ in. in diameter, the latter size occurring in the connections between eye-bars and girders. In length, they vary from $16\frac{1}{8}$ in. to $38\frac{1}{4}$ in. Those used in connection with the strand shoes have threaded ends while the remainder have blank ends. The pin-material is a basic open-hearth chrome-nickel steel, quenched and drawn. The lomas nuts are made of cast steel.

Chemical and Physical Requirements: The prescribed chemical and physical properties were the same as those for the heat-treated cable band bolts. They may be found in Tables 18 and 19 respectively. The number and method of selecting test specimens of this material had to be in accordance with specifications for quenched and tempered forgings of the American Society for Testing Materials (Designation A18-30).

Results of Tests: In Table 18 are recorded the results of the ladle and check analyses; in Table 19, the average physical properties.

TABLE 18.—AVERAGE CHEMICAL PROPERTIES OF THE HEAT-TREATED CABLE ANCHORAGE PINS—DETERMINED FROM LADLE AND CHECK ANALYSES

	C	Mn	P	S	Si	Ni	Cr
Specified (max.):							
Ladle.....	0.55	-----	0.04	0.05	-----	-----	-----
Check.....	0.605	-----	0.05	0.063	-----	-----	-----
Ladle*.....	0.37	0.68	0.020	0.022	0.18	2.04	0.52
Check:**							
Maximum.....	0.39	0.87	0.031	0.022	0.29	3.39	0.64
Average.....	0.36	0.71	0.018	0.021	0.24	2.04	0.47
Minimum.....	0.32	0.64	0.013	0.017	0.19	1.36	0.17

* 5 Melts.

** 10 Tests.

TABLE 19.—AVERAGE* PHYSICAL PROPERTIES OF THE HEAT-TREATED CABLE ANCHORAGE PINS—DETERMINED FROM MILL TESTS**

	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 2", per cent	Red. of Area, per cent
Specified (min.)	95,000	60,000	21.0	45.0
Maximum	106,000	71,000	27.5	62.3
Average	100,200	67,800	23.3	56.4
Minimum	95,000	62,500	21.0	46.8

*76 Tests.

**Specimen taken midway between center and outside of forging.

STIFFENING TRUSS ROCKER LINK PINS: There are 24 pins, two at each end of each truss. One of the pins connects the truss to the rocker link, and the other connects the latter to tower or pylon as the case may be. They have a central diameter of $13\frac{1}{2}$ in. while one end is $13\frac{7}{16}$ in. and the other $13\frac{9}{16}$ in. in diameter. They are about four and one-half feet long and are threaded at both ends. The material is a quenched and drawn chrome-nickel steel, part of which was made by the basic open-hearth process and the remainder by the acid process. The nuts are carbon steel forgings the material for which originated in both kinds of furnaces.

Chemical and Physical Requirements: For the pin material, the chemical and physical requirements were the same as for the heat-treated cable band bolts and cable anchorage pins. For the nuts, a material made in accordance with the specifications of the American Society for Testing Materials (Designation A18-30) Class F forgings, was prescribed.

Results of Tests: The average chemical properties of the link pins are given in Table 20; the physical properties, in Table 21.

TOWER SADDLE ROLLERS: There are 136 rollers. These members are 8 in. in diameter and 9 ft. $8\frac{3}{4}$ in. long. They are made of carbon steel which originated in both acid and basic open-hearth furnaces.

Chemical and Physical Requirements: The requirements were those specified by the American Society for Testing Materials (Designation A18-30) for Class F annealed forgings.

Results of Tests: The average chemical and physical properties of the material, together with specified requirements, are given in Tables 22 and 23, respectively.

SUSPENDER ROPE SOCKETS: Sockets are attached to the ends of the suspender and hold-down ropes. Those for the former are 8 in. in diameter and 14 in. long while the others are $8\frac{3}{4}$ in. in diameter and 18 in. long. The interior of these members is machined away to provide a conical-shaped hole into which the rope has been spread and then fixed in place with cast zinc. The sockets are made of basic open-hearth steel.

Chemical and Physical Requirements: Specification requirements were those prescribed by the American Society for Testing Materials (Designation A18-30) for Class C annealed forgings.

Results of Tests: Together with specified requirements, Table 24 records the average chemical properties of the material; Table 25, the physical properties.

TABLE 20.—AVERAGE CHEMICAL PROPERTIES OF THE HEAT-TREATED ROCKER LINK PINS—DETERMINED FROM LADLE AND CHECK ANALYSES

	C	Mn	P	S	Si	Ni	Cr
Specified (max.):							
Ladle.....	0.55	-----	0.04	0.05	-----	-----	-----
Check.....	0.605	-----	0.05	0.063	-----	-----	-----
Ladle*	0.34	0.69	0.028	0.029	0.19	1.34	0.69
Check:**							
Maximum.....	0.41	0.78	0.034	0.034	0.24	1.45	0.80
Average.....	0.37	0.68	0.030	0.030	0.22	1.35	0.68
Minimum.....	0.33	0.60	0.022	0.018	0.21	1.13	0.63

*9 Melts.

**15 Tests.

TABLE 21.—AVERAGE* PHYSICAL PROPERTIES OF THE HEAT-TREATED ROCKER LINK PINS—DETERMINED FROM MILL TESTS**

	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 2", per cent	Red. of Area, per cent
Specified (min.).....	95,000	60,000	21.0	45.0
Maximum.....	112,500	76,000	26.0	64.7
Average.....	102,300	68,800	22.8	56.5
Minimum.....	95,000	67,000	21.0	49.1

*15 Tests.

**Specimen taken midway between center and outside of forging.

TABLE 22.—AVERAGE CHEMICAL PROPERTIES OF THE ANNEALED TOWER SADDLE ROLLERS—DETERMINED FROM LADLE AND CHECK ANALYSES

	C	Mn	P	S	Si
Specified	-----	0.40 to 0.80	0.05 max.	0.05 max.	-----
Ladle *	0.45	0.70	0.020	0.028	0.25
Check: **					
Maximum	0.51	0.81	0.039	0.041	0.42
Average	0.45	0.71	0.016	0.028	0.25
Minimum	0.37	0.62	0.011	0.022	0.14

* 8 Melts.

** 25 Tests.

TABLE 23.—AVERAGE* PHYSICAL PROPERTIES OF THE ANNEALED TOWER SADDLE ROLLERS—DETERMINED FROM MILL TESTS**

	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 2", per cent	Red. of Area, per cent
Specified	80,000 min.	½ T. S. min.	$\frac{1,725,000}{\text{Ten. Str.}}$ (a)	$\frac{2,640,000}{\text{Ten. Str.}}$ (b)
Maximum	92,500	51,000	28.0	52.0
Average	86,100	46,300	25.5	42.8
Minimum	82,500	43,500	23.0	31.5

* 25 Tests.

** Specimen taken midway between center and outside of forging.

(a) Min., 19%.

(b) Min., 31%.

TABLE 24.—AVERAGE* CHEMICAL PROPERTIES OF THE ANNEALED SUSPENDER ROPE SOCKETS—DETERMINED FROM LADLE ANALYSES

	C	Mn	P	S	Si
Specified	-----	0.40 to 0.80	0.05 max.	0.05 max.	-----
Average	0.23	0.60	0.019	0.028	0.20

* 3 Melts.

TABLE 25.—AVERAGE* PHYSICAL PROPERTIES OF THE ANNEALED SUSPENDER ROPE
SOCKETS—DETERMINED FROM MILL TESTS

	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 2", per cent	Red. of Area, per cent
Specified	60,000 min.	$\frac{1}{2}$ T. S. min.	$\frac{1,700,000}{\text{Ten. Str.}}$ (a)	$\frac{2,700,000}{\text{Ten. Str.}}$ (b)
Maximum.....	67,000	42,000	35.5	63.5
Average.....	66,200	36,400	31.9	53.2
Minimum.....	65,000	33,600	27.0	41.9

* 10 Tests.

(a) Min., 25%.

(b) Min., 38%.

ALLEGHENY METAL: The lateral bracing system of the suspended structure is connected to the towers and pylons by means of pins made of Allegheny metal. These lateral pins, which are rectangular in cross-section, vary in sectional dimensions from 9 in. by 11 in. to 13 in. by 18 in., and in length from three to three and one-half feet. Other pins made of Allegheny metal, about 4 in. or less in diameter, are also used in the structure. In connection with these pins, rolled plate washers, also of Allegheny metal, are used. The material originated in a basic electric furnace and was heat-treated.

Chemical and Physical Requirements: The chemical composition of the material had to be in accordance with that outlined in Allegheny Steel Company's Bulletin 33 (1927). The physical properties had to conform to specifications of American Society for Testing Materials (Designation A18-30) for Class F forgings.

Results of Tests: Because of the limited number of tests involved, all results of physical tests and check analyses have been recorded in Table 26, together with the requirements.

SUSPENDER ROPE COLLARS: Each of the two pieces forming a collar is 1 ft. 11 $\frac{1}{2}$ in. long, 4 $\frac{1}{4}$ in. wide and 4 $\frac{1}{2}$ in. high. The half-collars are made from basic open-hearth carbon steel.

Chemical and Physical Requirements: The material was specified to be in accordance with specifications of American Society for Testing Materials (Designation A18-30) for Class C annealed forgings.

Results of Tests: The average physical properties together with the specified physical requirements are given in Table 27. The forgings were produced from two melts of steel of the following average ladle analysis: Carbon, 0.27; manganese, 0.44; phosphorus, 0.015; and sulfur, 0.031.

TABLE 26.—RESULTS OF CHECK ANALYSES AND MILL TESTS MADE ON THE ALLEGHENY METAL

	Ten Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 2", per cent	Red. of Area, per cent	C	Mn	P	S	Si	Cr	Ni
Specified	80,000 min.	1/2 T. S. min.	1,800,000 Ten. Str. (a)	2,800,000 Ten. Str. (b)	0.12 max.	0.50 max.	0.025 max.	0.025 max.	0.50 max.	12.0 to 16.0	---
Forgings:*											
13"x18" (T)	98,500	74,000	23.5	63.5	0.08	0.39	0.018	0.024	0.37	13.12	0.04
" (B)	98,500	82,000	24.5	64.7	0.08	0.38	0.019	0.024	0.36	13.30	0.04
11"x16" (T)	95,000	70,000	25.5	64.7	0.08	0.48	0.020	0.028	0.34	14.20	0.14
" (B)	95,000	70,000	25.0	63.5	0.08	0.49	0.019	0.029	0.33	14.10	0.14
9"x11" (T)	85,000	47,000	22.5	50.0	0.12	0.40	0.006	0.015	0.26	13.35	0.26
" (B)	86,000	60,000	27.5	68.8	0.10	0.40	0.006	0.015	0.26	13.32	0.20
8 1/2" dia.	95,000	70,000	25.5	66.9	0.08	0.42	0.015	0.011	0.35	12.75	0.05
Rods:*											
4" dia.	98,500	81,600	24.3	68.4	0.13	0.37	0.008	0.019	0.25	12.25	0.17
3" dia.	84,000	66,600	25.0	54.0	0.09	0.35	0.006	0.017	0.26	13.10	0.27
2 1/4" dia.	82,700	61,700	30.0	71.5	---	---	---	---	---	---	---
3/8" dia.	94,900	72,800	26.5	75.1	0.10	0.37	0.007	0.018	0.20	12.20	0.35
Plates:*											
6 1/2" x 1 1/2"	91,000	44,800	37.5	70.4	0.11	0.41	0.006	0.019	0.27	12.80	0.26
"	91,700	46,500	37.0	68.4	---	---	---	---	---	---	---
5 1/2" x 3/8"	90,700	43,500	39.0	64.8	---	---	---	---	---	---	---
"	90,600	47,600	35.0	70.4	---	---	---	---	---	---	---

(a) Min., 20%.

(b) Min., 32%.

(T) Top.

(B) Bottom.

*Physical test specimens for forgings and rods (except 3/8" dia.) taken midway between center and surface; for plates and 3/8" rod, specimen was of rolled thickness. Sample for analysis taken from physical test specimen.

TABLE 27.—AVERAGE* PHYSICAL PROPERTIES OF THE ANNEALED SUSPENDER ROPE
COLLARS—DETERMINED FROM MILL TESTS

	Ten. Str., lbs. per sq. in.	Yield Point, lbs. per sq. in.	Elong. in 2", per cent	Red. of Area, per cent
Specified.....	60,000 min.	$\frac{1}{2}$ T. S. min.	$\frac{1,700,000}{\text{Ten. Str. (a)}}$	$\frac{2,700,000}{\text{Ten. Str. (b)}}$
Maximum.....	71,000	42,600	36.0	61.2
Average.....	68,400	40,000	33.8	56.6
Minimum.....	66,000	38,500	31.0	52.8

*7 Tests.

(a) Min., 25%.

(b) Min., 38%.

OTHER MATERIALS

Among the different materials used in the bridge may also be found cast bronze, rolled brass, rolled aluminum, cast zinc, cast iron and malleable iron. Bronze is used principally for bushings and wearing plates; brass, for washers; aluminum for expansion joint covers; cast zinc, mainly for fillers between cable strands and for fixing rope ends in sockets; cast iron, for plugs and fillers; and malleable iron for wedges.

TABLE 28.—MATERIALS IN THE MAIN STRUCTURE, IN TONS

Materials	Anchorages			Tower	Cables	Suspended Structure	Total
	Tower	Pylon	Cable				
Structural Steel:							
Carbon.....	215	330	1,199	27,378	30	14,311	43,463
Silicon.....	-----	-----	-----	14,655	-----	9,010	23,665
Eye-bars.....	-----	-----	2,520	-----	-----	-----	2,520
Pins:							
Steel.....	-----	3	198	9	-----	9	219
Allegheny Metal.....	-----	-----	-----	-----	-----	6	6
Cable Band Bolts.....	-----	-----	-----	-----	102	-----	102
Rollers.....	-----	-----	-----	117	-----	-----	117
Cast Steel.....	-----	2	6	(a) 604	941	20	1,573
Sockets & Collars.....	-----	-----	-----	-----	111	-----	111
Cable Wire.....	-----	-----	-----	-----	21,597	-----	21,597
Suspender Rope*	-----	-----	-----	-----	1,211	-----	1,211
Hold-Down Rope**	-----	-----	-----	-----	14	-----	14
Hand Rope.....	-----	-----	-----	-----	29	-----	29
Wrapping Wire.....	-----	-----	-----	-----	334	-----	334
Bronze.....	-----	1	-----	6	-----	7	14
Aluminum.....	-----	-----	-----	-----	-----	5	5
Cast Zinc.....	-----	-----	-----	-----	43	-----	43
Malleable Iron.....	-----	-----	-----	-----	2	-----	2
Railing.....	-----	-----	-----	-----	-----	518	518
Field Rivets & Bolts (b)	-----	-----	-----	880	-----	272	1,152
Totals.....	215	336	3,923	43,649	24,414	24,158	96,695

* 12.0 lbs. per ft.
(a) Cable Saddles.

** 15.7 lbs. per ft.
(b) For anchorages included in carbon steel.



BOOK FIVE

FABRICATION. MEMBERS OF UN-
PRECEDENTED SIZE BUILT WITH
EXTRAORDINARY PRECAUTION
INSURED SUCCESSFUL EXECUTION
OF THE WORK IN ACCORD WITH
THE DESIGN.

FABRICATION

THE TOWER LEGS and the stiffening trusses, aside from the divisions of tower bracing, are the only components of the structure that had to be treated as units in the process of shop fabrication. The divisions of tower bracing, because of their simpler construction and comparatively smaller dimensions, were readily assembled at the shop as units. But such complete assembly of members either for a leg or a truss was impracticable on account of the length of these particular units. Even though facilities had been available for assembling a leg in full length, such work would have involved great difficulties because of its make-up. A leg has, between individual members, not only horizontal splices but also vertical joints, the horizontal splices being spaced in such a manner that members interlock with each other. How fitness of leg-members and truss-members was secured by assembling operations conducted on the parts of each unit is described on the following pages. In addition, a section has been provided in this report for reviewing the shop painting operations because of the unusual procedure that had been prescribed for doing this work.

TOWER LEGS

A tower leg is made up of plates and angles which have been combined to form a cellular design. Each cell is 42 in. square, with five exceptions; one, 42 in. by 126 in. in size; and four others, each 42 in. by 84 in. Considering the larger ones as multiples of the unit, there are at the base 103 cells of which 44, on the periphery of the leg, reach a height of about 23 ft. while others top off at various elevations until the number is reduced to 21 near the top of the leg.

The cells in a tower leg cross-section were grouped for purposes of construction into combinations of two or more, each resultant type being detailed by short length or column sections. The short outside combinations were known as the X-group. Those making up the remainder of the cross-section were designated by other letters as indicated in Fig. 9. The column sections of which the cell-groups are composed vary in length from 22 ft. 9 in. to 44 ft. 11 in. Cell-groups have, with one exception, two kinds of splices. One has been called the horizontal splice. It is the connection between sections in a cell-group and is staggered with respect to splices in other groups. (See Fig. 10.) The other, or vertical joint, has been termed the seam splice and it is the connection between cell-groups. The X-group, the exception, has only one splice or that along the vertical joints.

Fabrication of column sections took place in two shops. In both shops, however, the same general procedure was followed although details of the performance necessarily differed because of dissimilarity in equipment. The one main difference occurred with

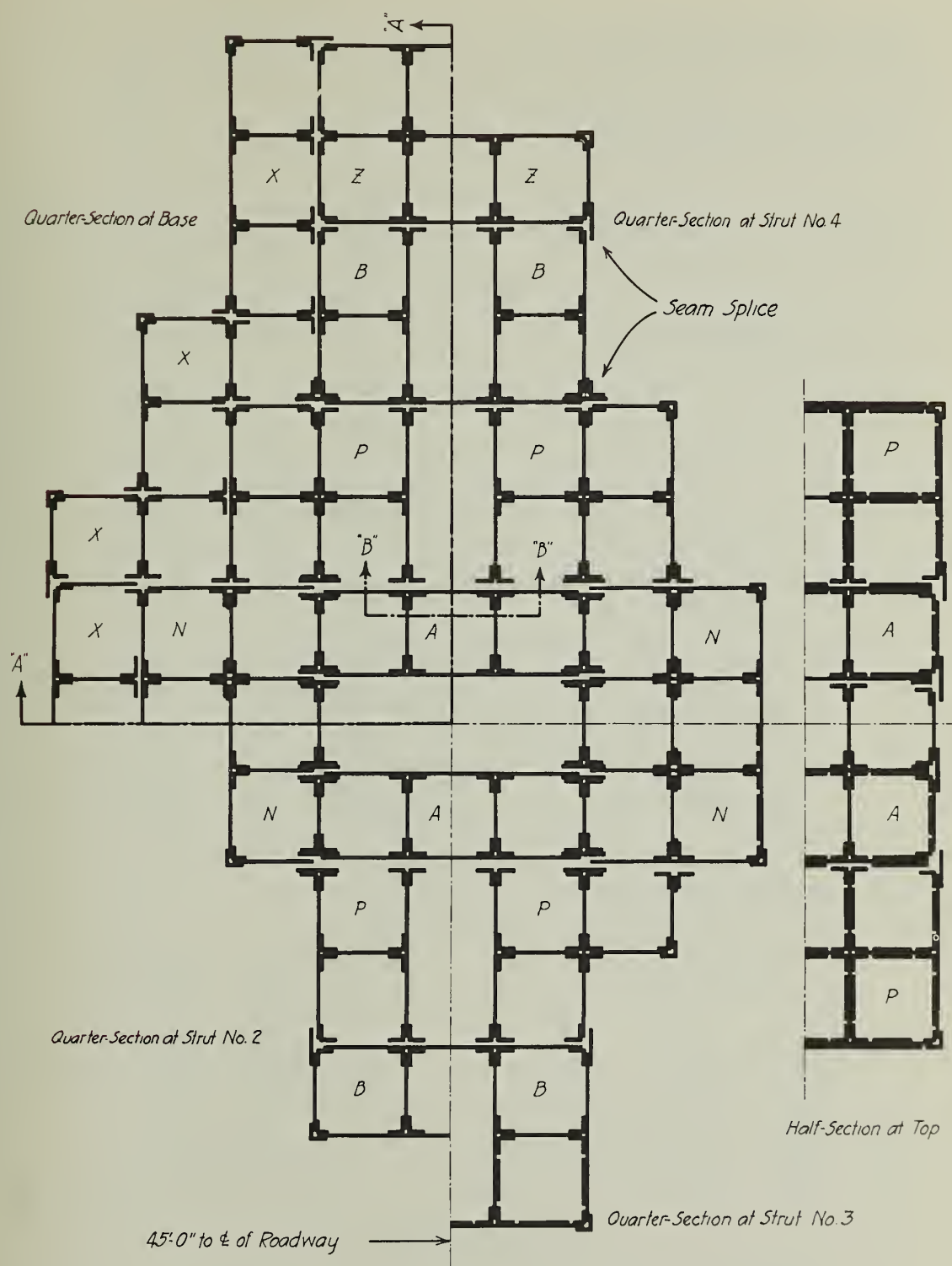


FIG. 9.—Grouping of cells for fabrication

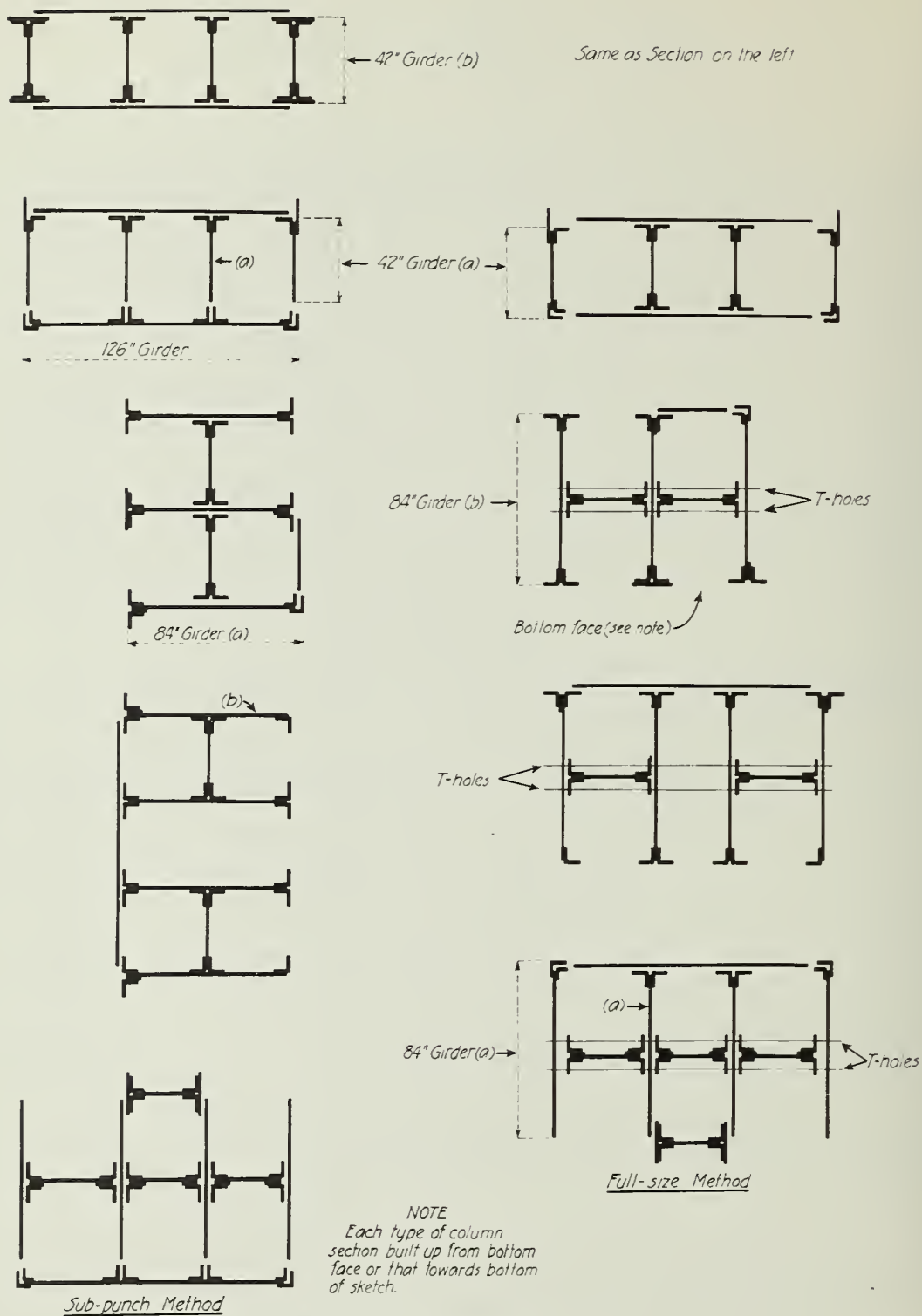
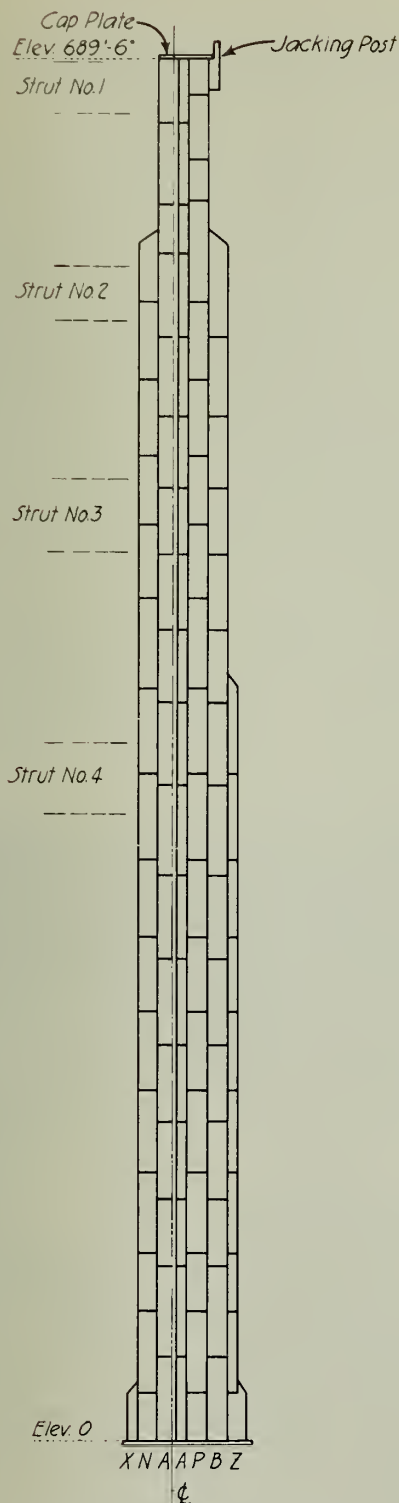


FIG. 10.—Two methods of boxing column sections



respect to the method of providing holes for shop-riveted connections. In one shop these holes were either sub-punched or sub-drilled, while in the other, many of them were drilled full size to template. Another difference occurred in that the latter shop used tack-welds for holding parts together in fitting and boxing operations. The way of performing the work in one will be called the sub-punch method; in the other, the full-size method.

For the purpose of this report, the fabricating procedures which were followed have been divided into three principal operations: Fitting, Boxing, and Assembling. These three divisions can easily be recognized from a study of cell-group cross-sections shown in Fig. 10 and their lengths indicated in Fig. 11. Under fitting operations is taken up the construction of the column section's composite parts; under the boxing operation, the work in connection with combining these parts into members or column sections; and under assembling operations, the work that took place in connection with making horizontal and vertical joints. Before proceeding with a description of these operations, however, there should be noted the work done in connection with templates because it influenced the details of fabrication.

Metal templates were made principally for use in drilling field-rivet holes in seam splices but the fabricator used them also in making shop-riveted connections. Originally, the seam splice holes were specified to

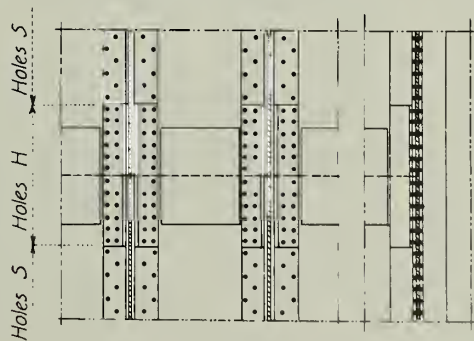


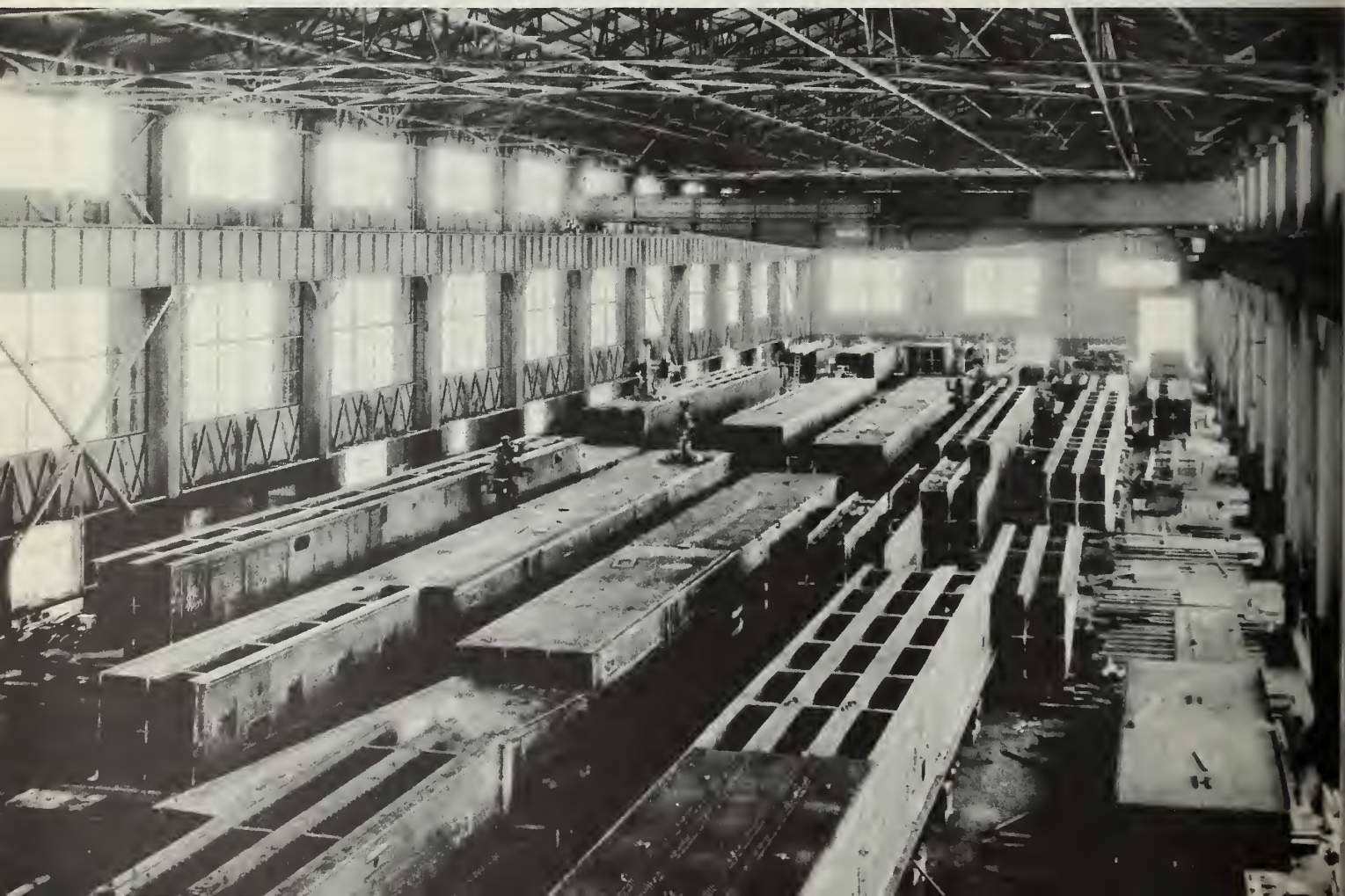
FIG. 11.—Elevation of tower leg showing horizontal splices and vertical seams

be reamed or drilled with members assembled at the shop or with members in place in the structure. But before work commenced on the towers, decision was made to drill the seam splice holes to full size through metal templates. The number of templates and consequently the amount of work involved in their setting would have been enormous had not special rules been instituted for preparing detailed drawings of the members.

So that the amount of shop work on the seam splice connections could be reduced, standard rivet spacing and exceptionally large templates (See Fig. 12) were used. The spacing of holes in the vertical direction was made $3\frac{1}{2}$ in., or multiples thereof, from the base of the leg to its top. Some exceptions occurred in detail material and at points near the top. Horizontal distances between gauge lines were made alike for each cell-side. In other words, if a template had been constructed as long as the leg and wide enough to fit over a cell-side, it could have been set on any cell-boundary for drilling or reaming both shop and field connections. Furthermore, any portion of its length could have been applied to a cell-side because the spacing of rivet holes in both directions was constant.

The working templates were made from a master template. They ranged in thickness from $\frac{3}{8}$ in. to $\frac{5}{8}$ in.; in length, from about 22 ft. to 46 ft.; and in width, from that which covered one seam splice to that which extended over all seam splices in a column section three cells wide. The master or full-sized template was $\frac{1}{2}$ in. thick and 45 ft. 6

Fig. 12—View in assembly shed showing two assemblies of center-span stiffening trusses. Drilling operations are proceeding on one while the other is being assembled. Starting panels for these assemblies are at the ends in the background



in. by 11 ft. 8 in. in out-to-out hole dimensions, or large enough to cover the longest section and the widest one. Its accuracy was checked by means of the first full-sized working templates struck from it. These plates had been drilled while stacked and this fact was utilized in the checking procedure by building up another stack with the same unbushed plates after turning them different ways in order to magnify errors in matching of holes. As they came from the original stack, one plate was laid on level skids; the next one, "flopped over" on the opposite side; the third, turned end for end; and the fourth, turned likewise but also flopped over. After this stack had been pinned through perfectly matched holes, a full-sized pin (about $1/64$ in. smaller than diameter of holes) was tried in every hole. The number into which the pin would not enter without being forced was practically negligible.

The possibility of distorting the working templates through bushing and handling operations also underwent investigation. After insertion of bushings, no appreciable amount of creep in the templates could be found by longitudinal and diagonal measurements made with a steel tape. Before putting them into use, they were reinforced with angles or small I-beams and in some cases fitted with eye-bolts so that they could be handled by special lifting apparatus in as flat a condition as possible. After one tower had been completed, the templates were checked to determine the effect of handling operations on their accuracy. The test consisted of matching holes in two members, those in one member drilled through a wide template and those in the other through a narrow template which contained holes for only one splice. The result showed that mismatching occurred in 5% of the holes and amounted to a maximum of about $1/32$ in.

FITTING OPERATIONS: The composite parts, fitted or otherwise prepared for the boxing operation, were plates, girder-like pieces and horizontal diaphragms. Plates are of one-, two- or three-cell width; some have detail and reinforcing material attached to them. The work of preparing these parts for the next step in fabrication was confined principally to punching and drilling operations. The girder-like pieces are also of one-, two- or three-cell width and are referred to as 42-in., 84-in. and 126-in. girders.

It is apparent from the design that a column section had to have accurately made field connections as well as abutting joints if it were to interlock with adjacent ones in a way that would require no extra work in the field. The section, therefore, had to be straight and square, and also correctly dimensioned in respect to separation of webs and location of field holes. Because fitness of a column section depended upon the dimensions of its composite parts, the fabricator strove to obtain true horizontal diaphragms, and girders with exact depth and straight parallel flanges.

Fitting 42-in. and 84-in. Girders: The fabricator employed jigs for making these parts straight and true. The make-up of some girders prevented their fabrication in such a form, but those which served to control the "packing-out" feature in cross-sectional dimensions passed through the apparatus. Typical examples of the first-mentioned parts

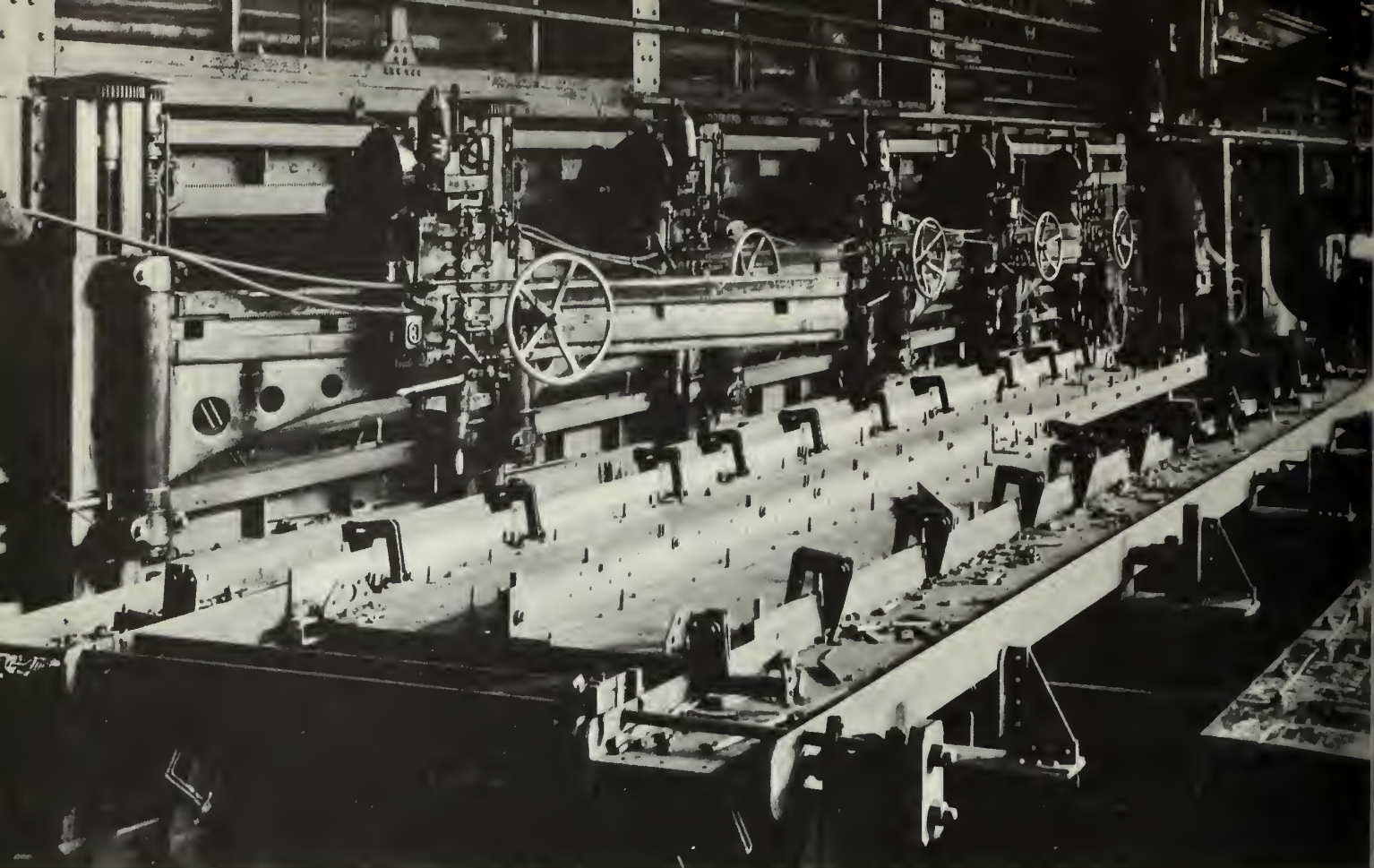


Fig. 13—The 84 in. jig used in the sub-punch method. A special girder has been fitted in it

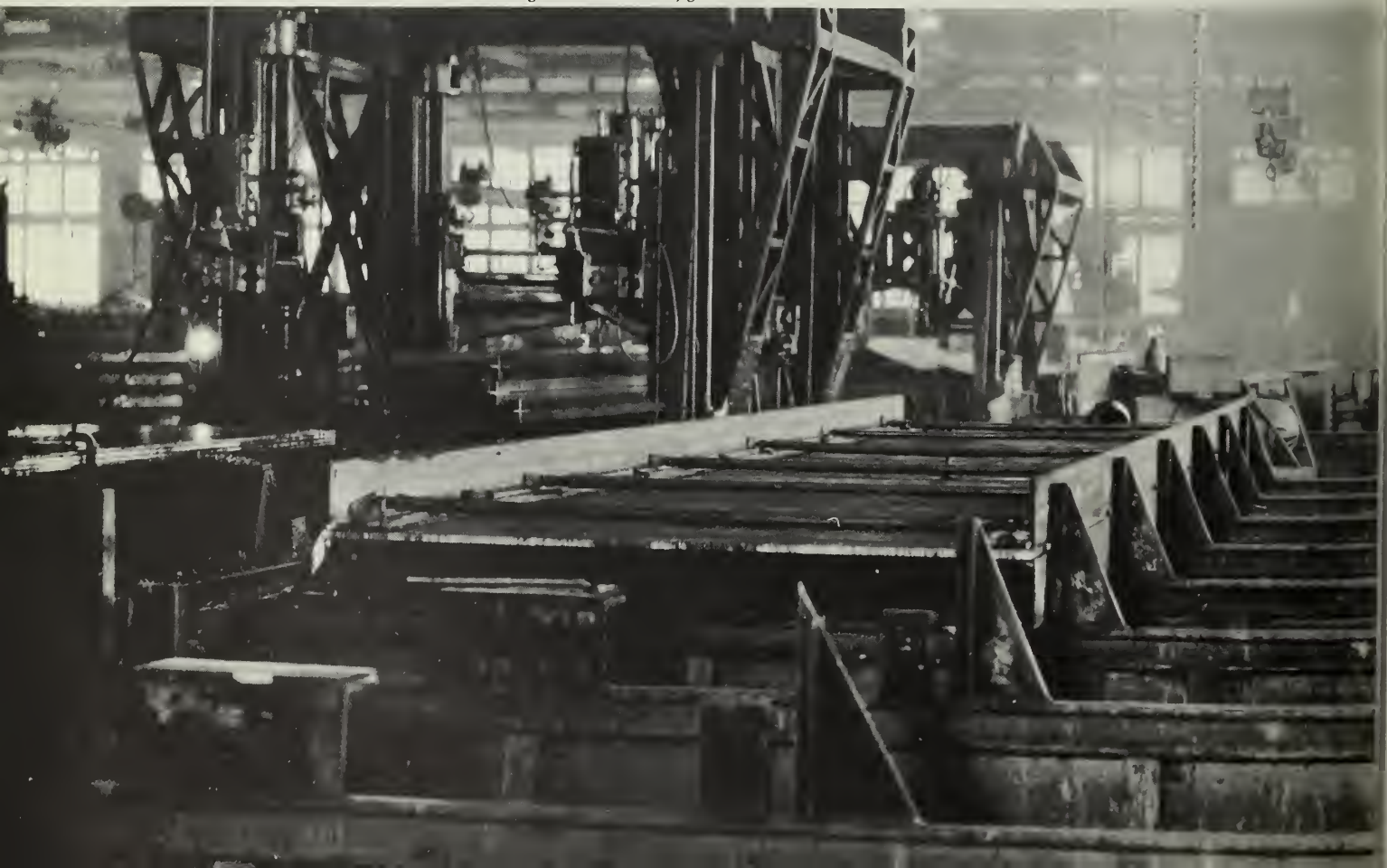


Fig. 14—The 84 in. jig used in the full-size method

are noted as (a) and the others as (b) in Fig. 10. The jig was used for fitting the web plate and flange angles.

The jigs (one for each size of girder) had to be constructed so as to permit manufacture of girders which differed in depth between flange angles on account of the cell-side thickness and the presence of what may be termed cover plates. A jig in its simplest form consisted of two lines of stops (one along each girder flange) attached to level skids which had a separation of three or four feet. One type of jig (Fig. 13) had two planed beams for stops, the planed surfaces of which provided contact for girder flanges; the other (Fig. 14) had individual uprights welded to each skid. In the first-mentioned scheme, one beam could be moved for altering the girder-depth. In the other, the jigs had fixed widths for the deepest girders so that fillers had to be used along one side for controlling the member's depth.

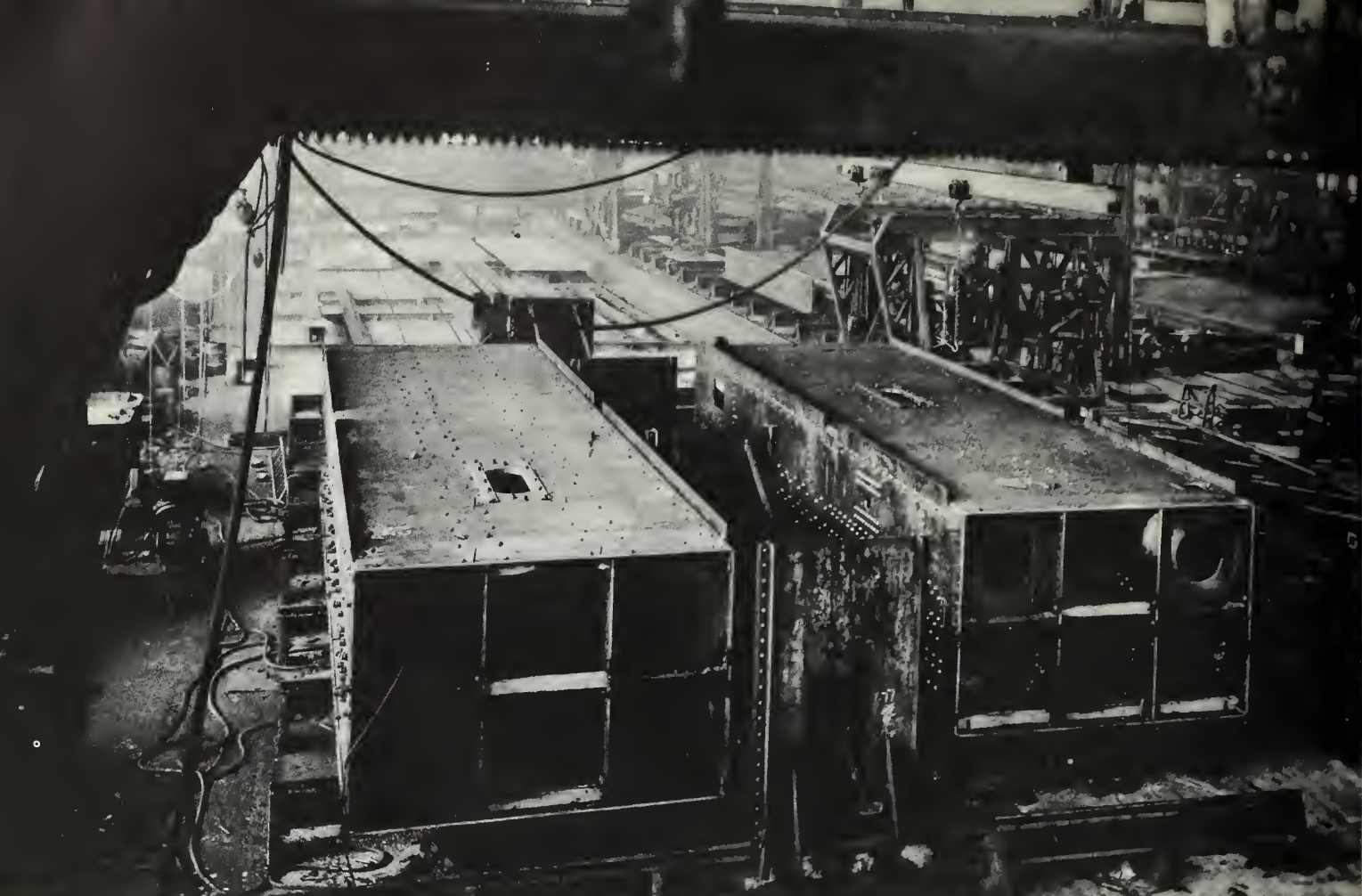
Fitting 126-in. Girders: Location of angles on the plate was an important consideration in fabrication of the girders because the position of angles affected the width of cells which had to be square. With this in view, the fabricator set up corresponding rows of U-shaped clamps on skids in such a way that when angles were dropped into them the distance between sets of angles was correct. The concluding step of the operation consisted of placing the plate on the angles and fastening the parts together.

Horizontal Diaphragms: These diaphragms, being the parts upon which depended the squareness of column sections, were planed or milled on the web edges. To obtain their maximum efficiency, which occurred when the web was pulled to bearing with the cell-side, the connection angles were set back slightly from the milled edge.

BOXING OPERATIONS: The boxing operations consisted of assembling the composite parts of a column section (See Fig. 15), squaring the section and, finally, riveting the parts together.

In Fig. 10 two methods of boxing sections are shown. One was followed under the sub-punch method and the other under the full-size method. In the latter scheme, holes of shop-riveted connections marked "T" were made full size from templates. In these connections the holes in the 84-in. girders were drilled through a template, two cells wide, set in relation to the flange angles; in the 42-in. girders they were drilled from a narrow template or one as wide only as the flange. The latter template required special care in setting operations because of sweeps in both template and girder. To avoid inaccuracies that might arise from this combination of factors, template center-line was made to coincide with web center-line by adjusting set-screws which had been attached to both edges of the template.

The important matter of squareness in cells of column sections was controlled by the use of required and "dummy" diaphragms as well as jacks. To gain full advantage of the regular diaphragms, or, in other words, to obtain the best possible bearing between diaphragms and cell-side, the diaphragm angles were brought into full contact with the cell-web by means of complete bolting. At both ends of a section, where diaphragms



View of column-sections at boxing skids. One column-section has been riveted and is ready for the milling operation; the other is in the reaming and drilling stage

had not been provided, temporary plates, squared by means of a planer, were used as well as jacks. Trammeling operations for the purpose of checking intersections of cell-web center-lines on ends of sections took place before the member was riveted and again before it was milled.

ASSEMBLING OPERATIONS: Two kinds of assemblies took place during the course of the work. One was a line assembly of each cell-group (Fig. 11), the other, an assembly of each leg top with part of the cap plate.

Cell-Group Assemblies: With respect to assembly of column sections for a tower leg, specifications originally provided that the horizontal splices be reamed or drilled with parts assembled at the shop, and, as already stated, that the seam splices be reamed or drilled either at the shop with parts assembled or in the field after the members had been erected. The requirement relating to the method of making the seam splices, however, was modified by the Engineer. Under this modification the fabricator was to make these joints by means of metal templates at the shop; the accuracy of results from the entire procedure had to be checked by means of a trial erection of at least 60% of the cell-groups of the base tier of one leg. This demonstration of the sufficiency of methods was required to be done before continuing with fabrication of other cell-groups.

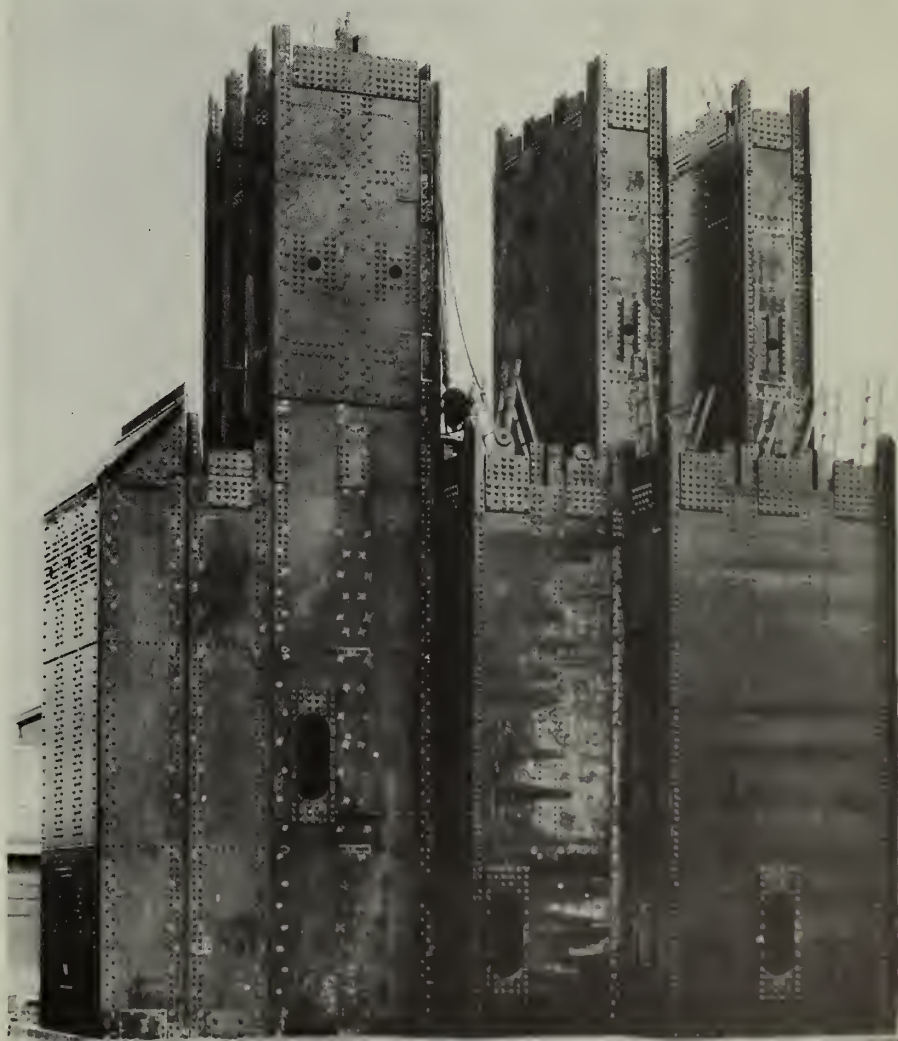
The program for the assembling operations required, therefore, the work of assembling horizontal splices for drilling and reaming, and that of drilling the seam splices. Fig. 11 shows the typical lines of cell-groups that were assembled. A line started with the base section and continued until it topped out or merged with another as in the case of the P-lines which became two. Assemblies consisted of three to eight column sections in a line. In the sub-punch method the column sections were completed to such an extent before assembling that after they were placed in the assembly line only the horizontal splices had to be reamed to complete reaming and drilling operations. The full-size method followed a scheme in which most of the seam-splice holes had to be drilled with members assembled.

For setting seam-splice templates the same procedure was followed in both the sub-punch and full-size methods. All templates were set with respect to the milled joints, the elevations of which (distance from base of leg) were recorded as a part of the procedure.

In the sub-punch method all field holes, excepting those in horizontal splices, were drilled before assembling the members. The seam-splice holes noted as holes "H" in Fig. 11, or those occurring in the horizontal splices, were reamed or drilled through a template with joints assembled. This template, called a "jumper" template, was doweled to some of the full-sized seam-splice holes (noted as holes "S" in Fig. 11) which had been previously drilled on either side of the milled joints.

In the full-size method most of the seam-splice drilling took place on assembled members. Only those seam-splice holes in the bottom side of the member, or that face which rested on skids in the assembled line, were drilled either before splicing or after disassembling. This sequence of handling the bottom face affected the method of setting jumper templates for holes "H". When the work of drilling holes "S" was done before assembling them, the template was doweled to these holes, but when holes "S" had to be drilled after disassembling them, the jumper

Trial erection
assembly

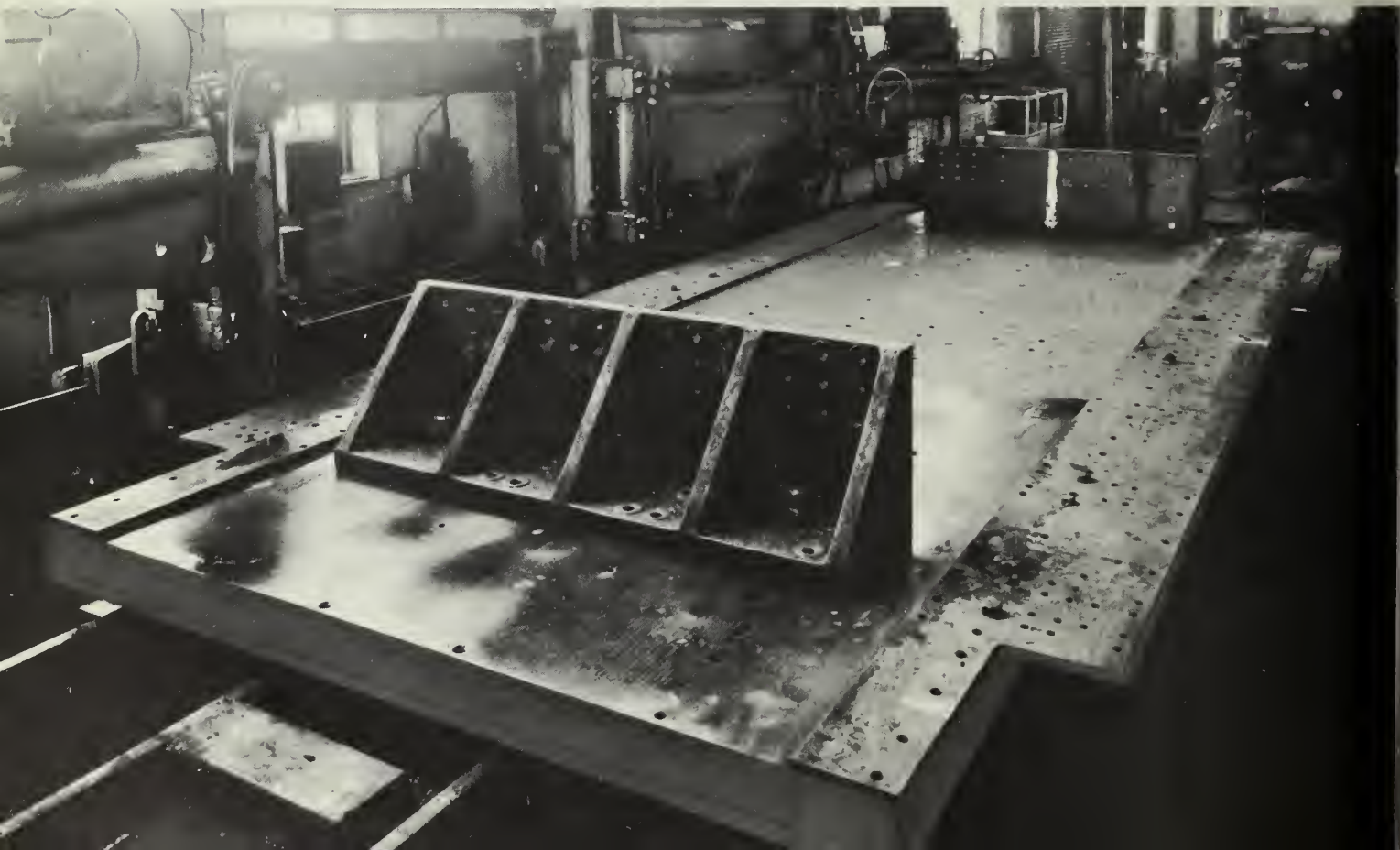


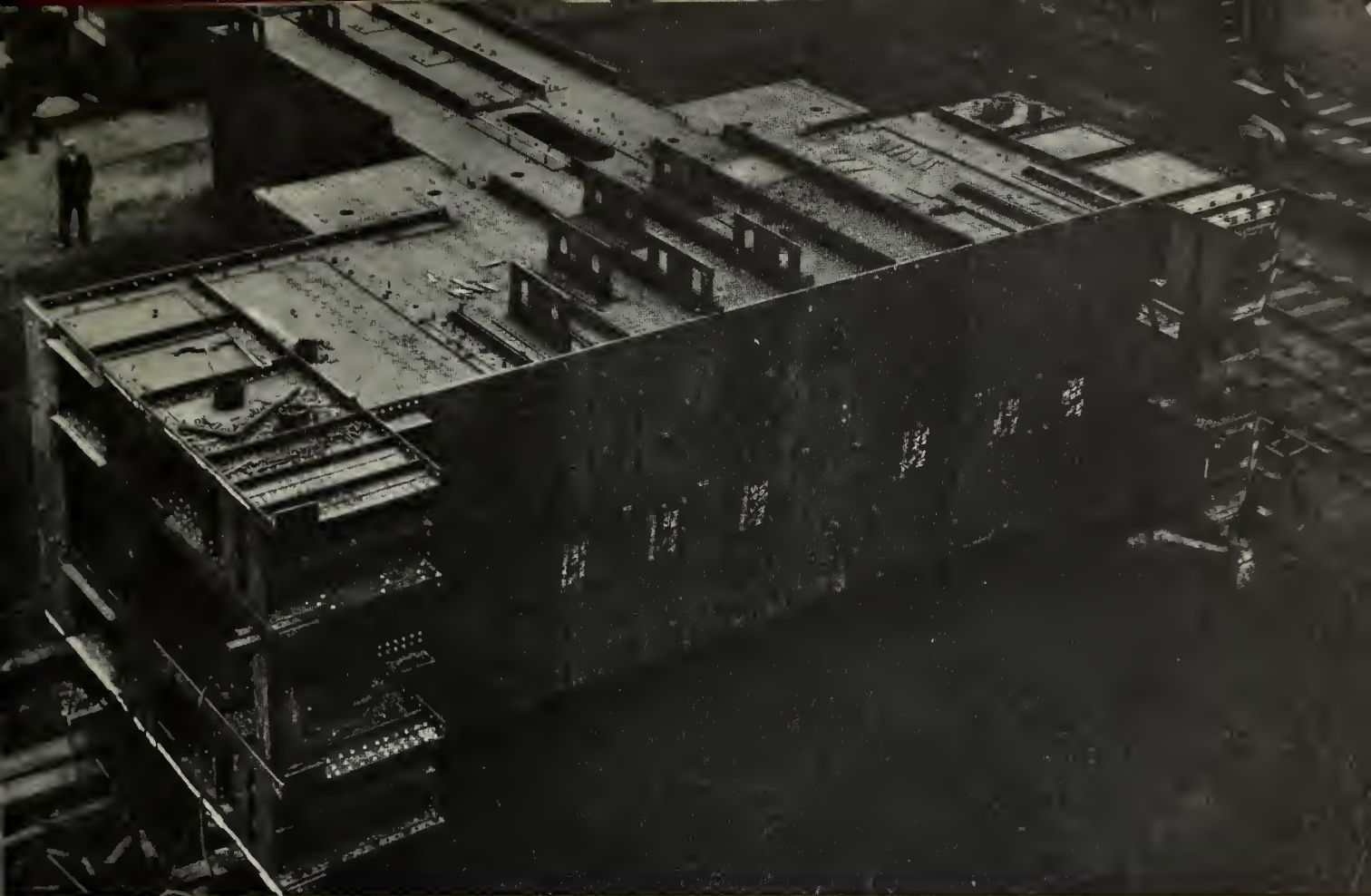
template was set with respect to the milled joint. In the latter case holes "H" as drilled through the jumper template were checked when the full-size template was applied to bottom face of member.

Jacking Post Assemblies: Each of these assemblies consisted of the four column sections which form the top of the leg, the jacking posts (which were temporary members required in connection with adjustment of the cable saddles) and parts of the cap-plate unit. Before assembling these parts, the column sections had been completed except for drilling holes in the cap-plate and jacking post connections. In the meantime also the jacking posts' composite parts had been fitted, and the cap-plate unit had been completed. The reason for making the assemblies was three-fold; (1) to insure flushness between post shelf and column top, (2) to obtain good bearing between bottom end of post and column side, and (3) to locate accurately the holes in connection of cap-plate unit to leg top.

Attainment of requirements in connection with jacking posts necessitated a program wherein the posts could be fitted by assembling their composite parts on the column section. By this arrangement the parts of the posts were adjusted to the desired position both with respect to bearing at the cap-plate and at the section's side. To obtain good bearing at the latter point, grinding of both post and column-section parts was required in some cases. After the requirements had been met, and before removing a post from the assembly, its parts were tack riveted together so that they would not lose their relation with respect to each other during the disassembling operations.

Tower cap-plate completely assembled in the shop





View of assembled tower-leg top showing a jacking post on either side and the fillers of a cap-plate

STIFFENING TRUSSES

The stiffening trusses are of the Warren type and are used in spans of deck construction. Both the center- and side-span trusses are cambered. At each end they connect, by means of a pin, to a rocker link which in turn is pinned to tower or pylon as the case may be. Between centers of pin holes in the center-span trusses there is a distance of 4143 ft. 11 $\frac{3}{8}$ in., or 166 panels of 24 ft. 11 $\frac{9}{16}$ in.; the like dimension in the side-spans is 1097 ft. 5 $\frac{3}{4}$ in., or 44 panels of 24 ft. 11 $\frac{5}{16}$ in. The trusses are suspended from the cables (at alternate panel points) by means of ropes which pass through the top chord and connect to verticals.

In the work of making the individual truss members the fabricator had to assemble them. He followed this course not only for obtaining accurately made field connections but also for securing proper adjustment of the members' detail parts. This way of fabricating members proved to be a very convenient one because it eliminated, before assembly, the work of making otherwise identical members distinctive by a small bevel in field connections brought about because of the camber, and thereby avoided the errors with consequent delay and loss of material that could have arisen from placing members in wrong locations in the assembly. Such a procedure also permitted the fabricator to defer attachment of detail parts until the member had been placed in the assembly when this material could be set with assurance of correctness in the fit.

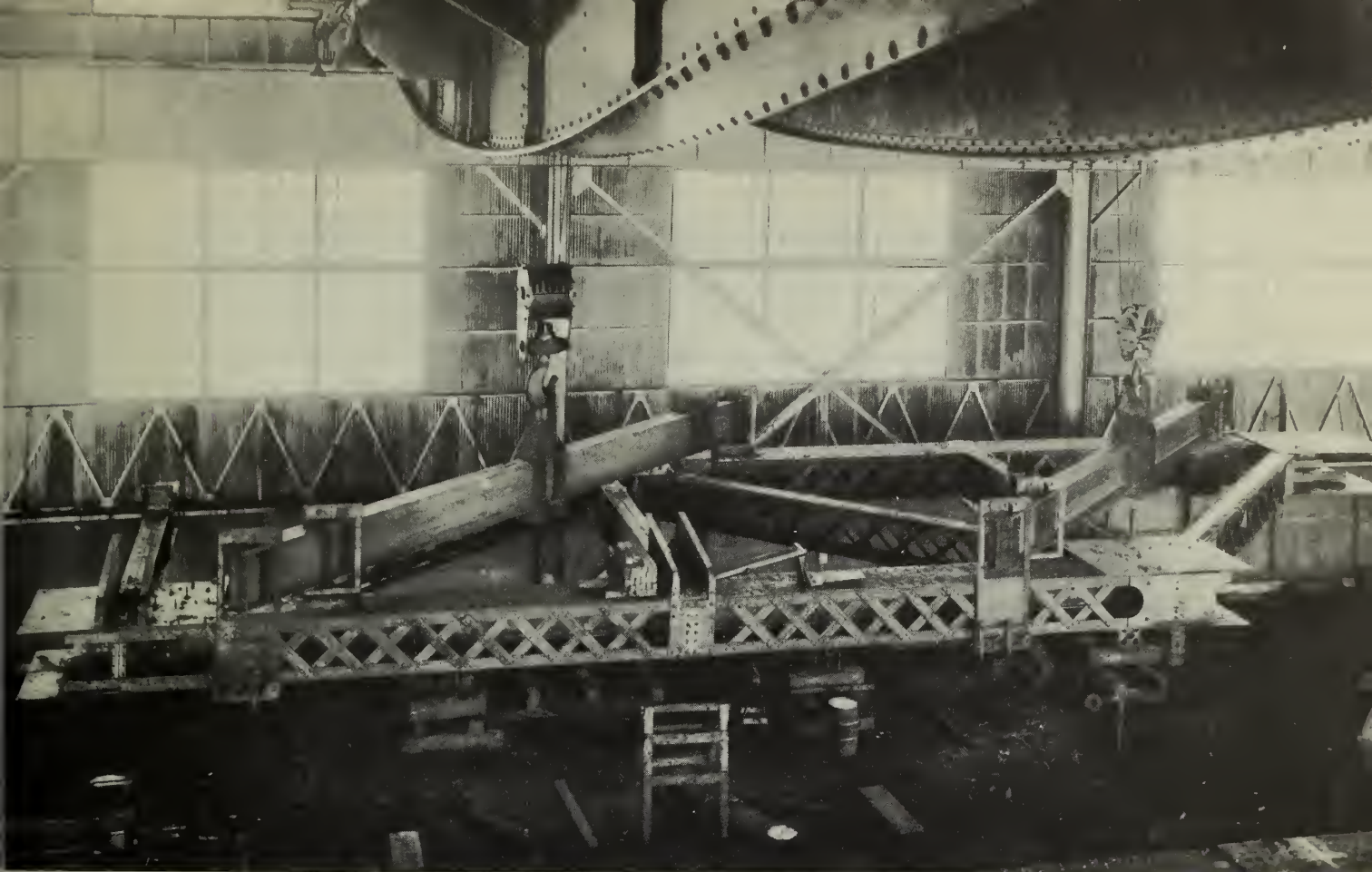
Because of the program adopted for their fabrication, the truss members before assembly consisted of little more than their principal elements. No complicated operations were involved in their fabrication because the sections were simple. Since the assembling operations governed chiefly the work of making accurately fitted members, they, as well as the measuring operations are described below.

ASSEMBLING OPERATIONS: All members and the details needed to make each one complete were brought together in the assembly. These details occurred at the top-chord splice joints and at the bottom-chord end of each vertical. On the top-chord they consisted of gussets, angles and small plates, most of which were eventually shop-riveted to the chords; on the bottom-chord, of similar material and brackets in addition, most of which had to be attached to the verticals. The details for the verticals were tack-riveted while assembled in order to maintain their true relations. Other details, such as those at the top-chord splice and some at the bottom-chord splice, were left loose because they had to be removed and then cleaned before riveting them to members. All these details, however, could not be assembled at once because some pieces had to be left off to permit the making of camber measurements, and others, such as brackets, could not be set until holes which they had obstructed had been drilled.

Assembling work was begun at one end of a truss and continued to the other in a succession of assemblies. Specifications required at least eight panels to constitute an assembly and that the last two panels (consisting of at least two chord sections, two verticals and three diagonals) be held intact or reassembled in exact condition for starting another assembly. The fabricator used longer assemblies because space for the work permitted the extensions. Also, he depended on intact starting assemblies for continuations. In the case of two side-span trusses the space was sufficient to continue with assemblies in succession without moving back any starting panels. Each of the other two were assembled under conditions which required only one retracement of the starting panels. The center-span trusses were assembled in units of nine panels and, therefore, required forty assemblies for the two trusses. The starting panels in all cases were checked, before and after relocation, with steel tapes by means of points center-punched on the two chords.

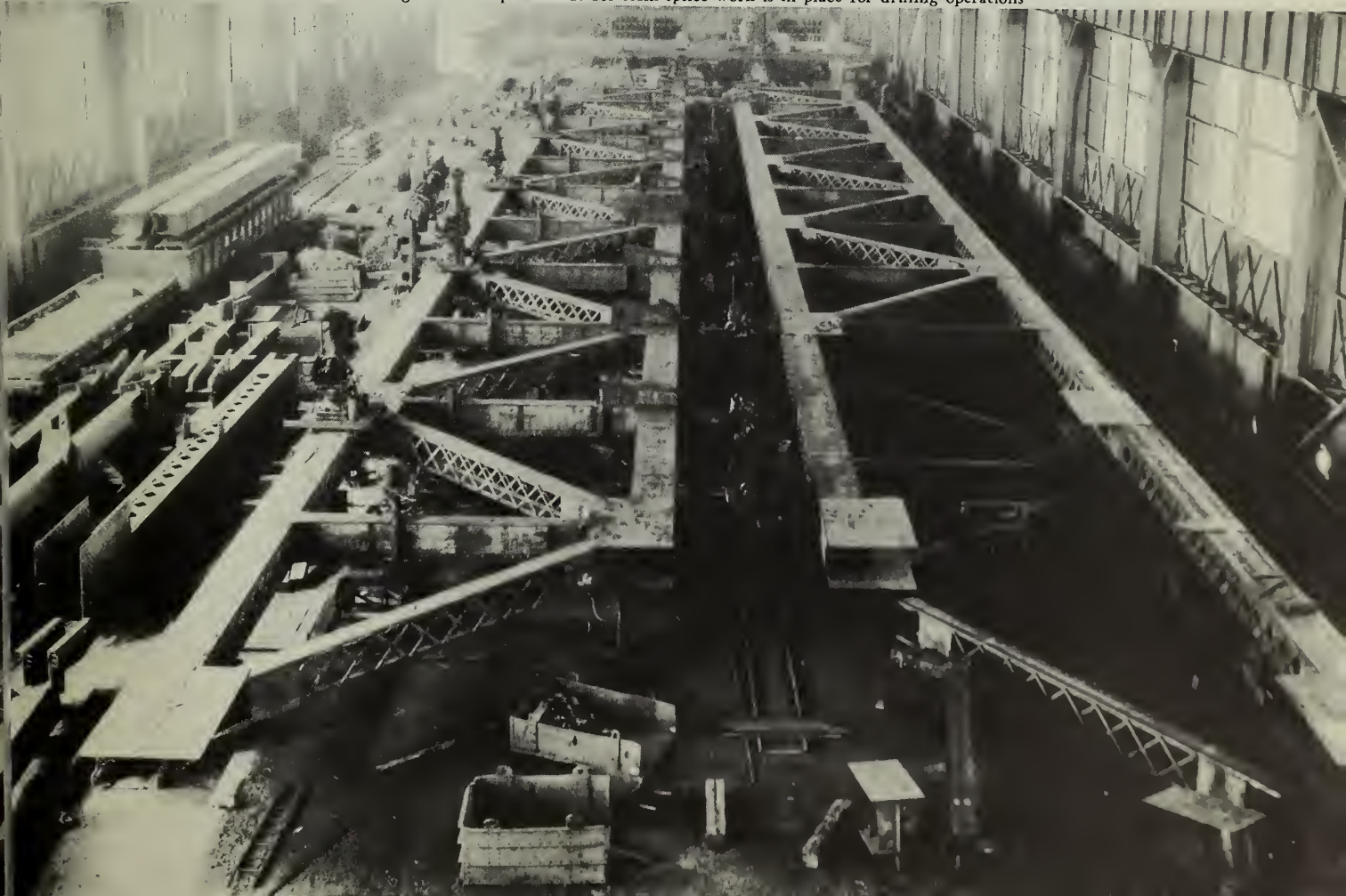
The trusses were assembled with the roadway side as the top face because most of the work took place on it. At the top-chord panel points, templates had to be set for floorbeams and lateral bracing connections; at bottom-chord panel points, for knee brace connections. The bottom face, or that side in contact with the skids, required little template work.

MEASURING OPERATIONS: The trusses which extend from pylon to pylon and are supported on the suspender ropes, have been designed to follow a curve, the chord of which could be passed through the end panel points at the shore-ends. In fabricating the trusses to the desired curvature, the assumption was made, of course, that their supports would be located at the specified elevations.



Above: Starting panels of a center-span stiffening truss assembly, picked up for removal to a new location

Below: View in assembly shed showing column-sections in various cell-group assemblies. In the foreground the largest size template made for seam-splice work is in place for drilling operations



In the assembling work, configuration of the truss was determined by ordinates from the chord of the curve. To simplify these operations, measurements were made from a line laid down, parallel to the chord, on each assembly. This base line was established, by means of elevations, from the end of panel points of an assembly. One end was fixed at the lower panel point and the other was located to the same elevation by dropping down a distance equal to the difference in elevation between the two panel points. Since the truss-verticals had to be set perpendicular at the chord line, the base line was made as one side of a rectangle which had a length of at least 8 panels and from which accurate right-angle measurements could be made.

The procedure for bringing an assembly into proper configuration was one of trial and error, but it consisted of the following fundamental steps: After the splices had been made tight and the web members loosely bolted in place, the panel points were established on top and bottom-chords. The two ends of the base line were then fixed, as stated, and a piano wire, stretched by hanging weights on both ends, was set over them and high enough to clear assembled members between the two panel points. The curve of the top-chord was first made to conform to the theoretical ordinates between wire and alternate panel points, or those where splices occur. The bottom-chord was next brought into line with respect to the top chord by maintaining the required depth of truss. At this stage of the work, the verticals which had on them the suspender rope connections were set so that in each case the connection had the correct relation to panel point. After the curve of the top-chord and the depth of truss had been assured, the location of the base line was scribed at each vertical, the wire moved to the bottom of the truss and made parallel to the base line and this location of the wire then scribed at each vertical. Holding these parallel lines as two sides of a rectangle the other two sides were established by dividing the rectangle into two smaller ones and then making their diagonals equal in length. From a short side of the rectangle, panel lengths were measured along both long sides of the rectangle. Through the two points thus established at each vertical, a chalk line was snapped across the truss' depth. This line provided a reference for setting the vertical and also permitted a check on the relation, length-wise of truss, between top-chord and bottom-chord panel points. After completion of measurements, templates for drilling field connections were set in relation to panel points and the various lines.

SHOP PAINTING

Shortly after fabrication of members for the suspended structure had gotten under way, specifications relative to paint and painting procedure were completely revised. Prior to the change, one coat of paint composed of twenty-five pounds of red lead to a gallon of linseed oil was applied to members. For the remainder of the work, or on members for the suspended structure, specifications required from one to three coats of a paint made up with 32% synthetic vehicle and 68% red lead. The one coat of this paint was specified for only the top flange of those stringers under the roadway curbs and

for field connections; two coats for all other parts of members; and a third coat for inside surfaces of box-like members. The field connections had to have this one coat of synthetic-vehicle paint covered with one of a preparation known as blue lacquer. No surfaces shop-riveted into contact had to be painted.

Before the work of painting could be commenced under the newer requirements, about 50% of the suspended-structure material had been fabricated but not painted. This material was stored, and, in the interim, a heavy coat of rust had accumulated and the scale had become loose over large areas. The work of cleaning the members, therefore, became a major operation, in fact it involved more labor than did the actual work of applying the paint.

Cleaning Operations: At the start of cleaning operations the ordinary scraper and wire brush were used for cleaning surfaces. This method was effective though time-consuming. To speed the work, the fabricator devised special cleaning tools. These tools consisted of circular wire brushes inserted in hand-controlled, high- and low-speed machines. The high-speed ones were used generally on flat surfaces or areas free of rivet heads and open holes because these details tore bristles from the brushes, and therefore, rapidly reduced the tool's efficiency. The low-speed machines proved to be a satisfactory means of cleaning areas involving these details although hand scrapers and hand brushes were employed as well.

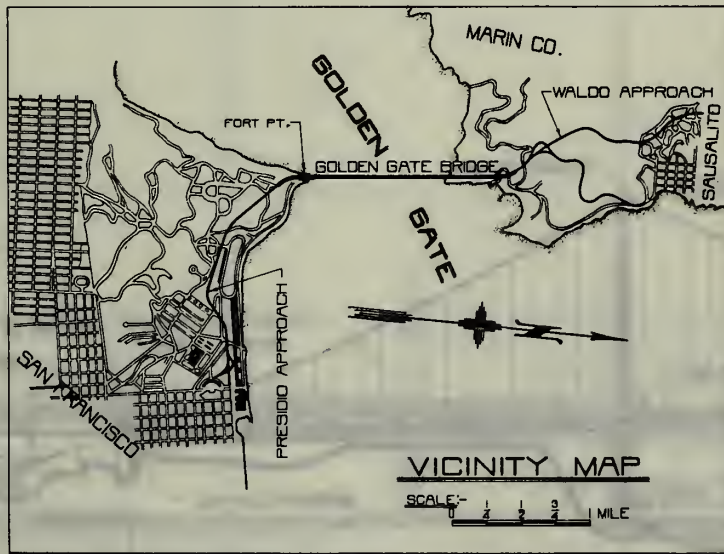
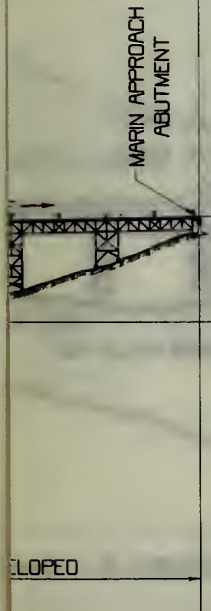
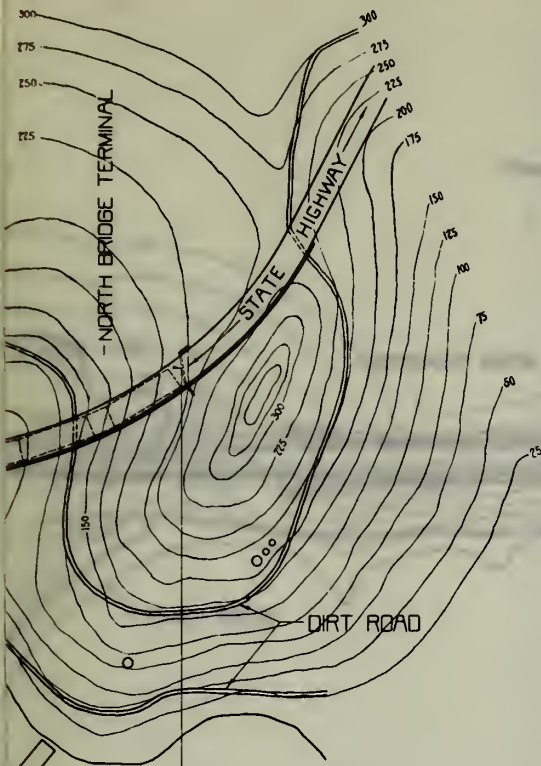
The cleaning procedure in general was conducted along the same lines on all members. The members were dampened or sprayed with solvent to loosen dirt and scale, buffed with brushes, subjected to an air-blast and, finally, wiped with waste that had been dampened with solvent.

Painting Operations: Requirements permitted the use of both the spray and the brush for applying the paint. However, at the very beginning of the paint work, different procedures in applying the paint to rivet heads and surrounding areas were tried because of difficulties which arose in properly covering these surfaces. After a comparison of results had been made, the following painting procedure was adopted: (1) rivet heads brushed with priming coat, (2) surfaces sprayed with same coat, (3) sprayed with second coat, and (4) sprayed or brushed with third coat. In the case of miscellaneous or small members, both the spray and the brush were used in the last three steps of application.

Results: From the standpoint of appearance of the paint work, the results proved to be very satisfactory but not without first overcoming the inefficiency of the spray-method for applying the paint. Poor workmanship resulted generally on this account around rivet heads, in confined spaces and on surfaces which were not sprayed in a downward direction. Directing the spray upward never produced satisfactory results, and not always when directed horizontally, because the operator, on account of the fog set up, could not follow easily the results of his work, particularly at ends or edges of materials. As a result, the operator tended either to flood or barely cover surfaces with

sprayed paint. The disadvantages which were revealed during the course of spraying operations were overcome by brushing as a preliminary step before spraying, by turning members to permit downward spraying and by developing ways of applying this paint with a spray. For example, in the case of some floorbeams and chord sections which presented surfaces for upward spraying operations because the members could not be turned conveniently, such underside surfaces were brushed either partly, as around rivet heads, or completely, before being sprayed with a like coat. On members which presented vertical surfaces for spraying, a technique consisting of two steps was finally developed which produced excellent results. In the first step the surface received a light spray. By the time the surface was covered, the quick-drying quality of the paint permitted the operator to take the second step, or to spray the surface a second time. This routine allowed the operator to cover surfaces thoroughly without piling up the paint.

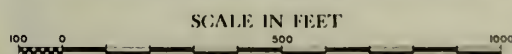
PLATES

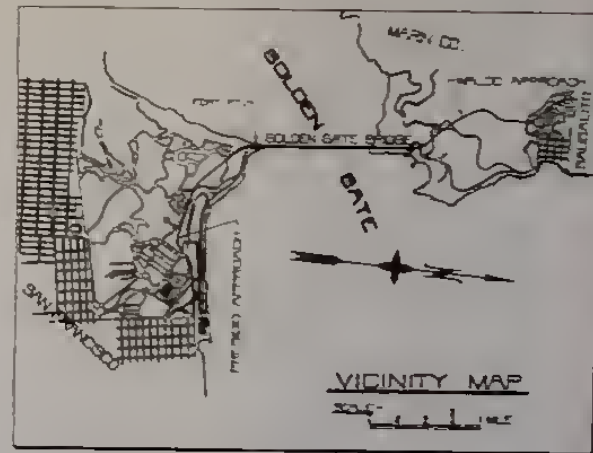
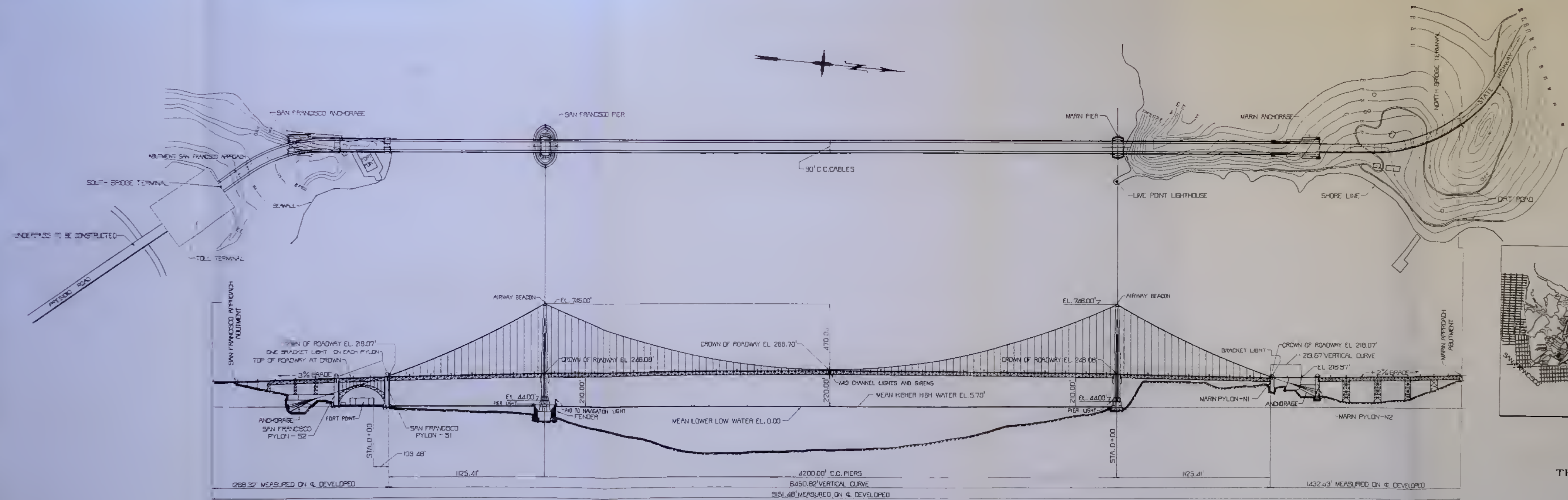


THE GOLDEN GATE BRIDGE SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

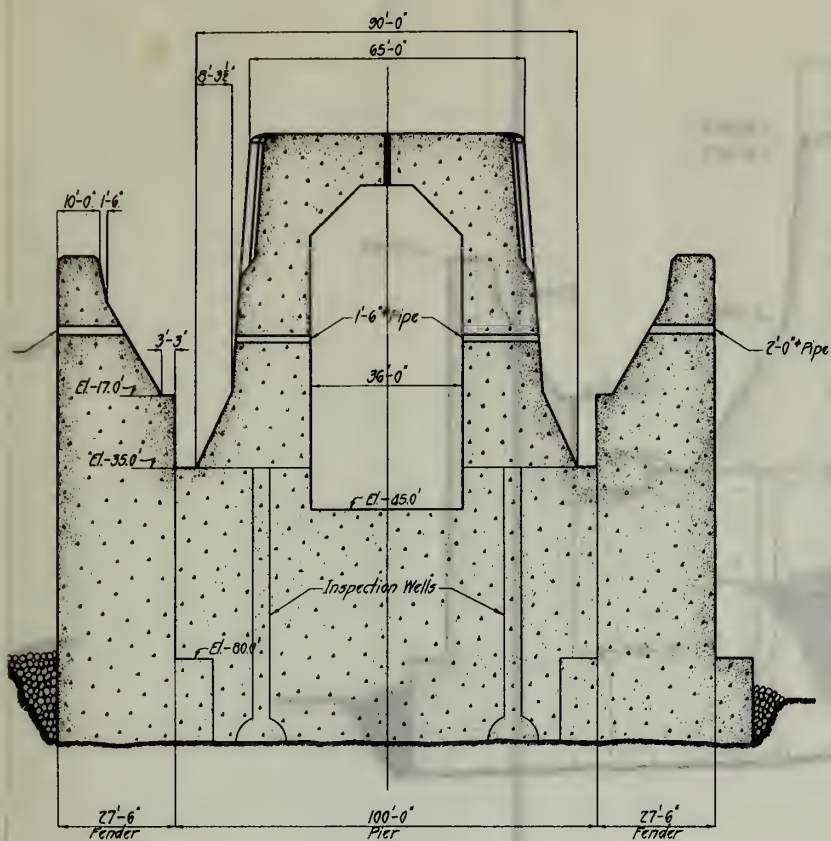
MAIN BRIDGE PLAN AND ELEVATION





THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.
PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937
**MAIN BRIDGE
PLAN AND ELEVATION**
SCALE IN FEET

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SECTION 'B-B'

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+ for Pier to Elev. -35.0'
other concrete poured in the dry.

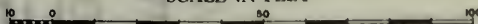
THE GOLDEN GATE BRIDGE

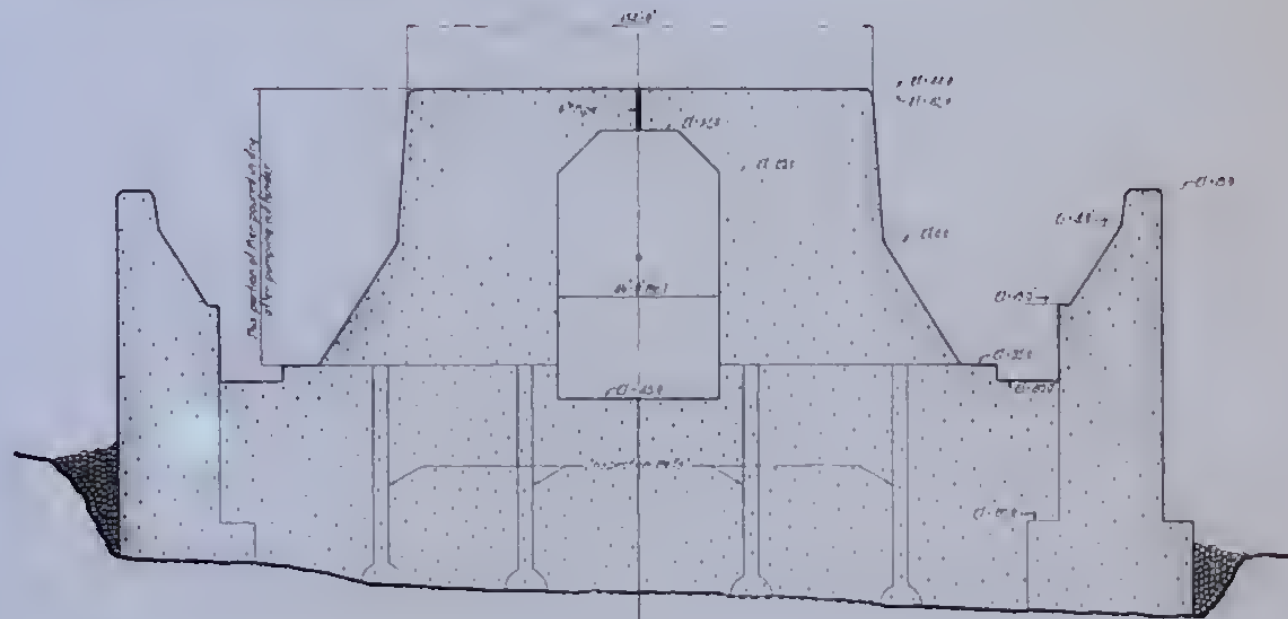
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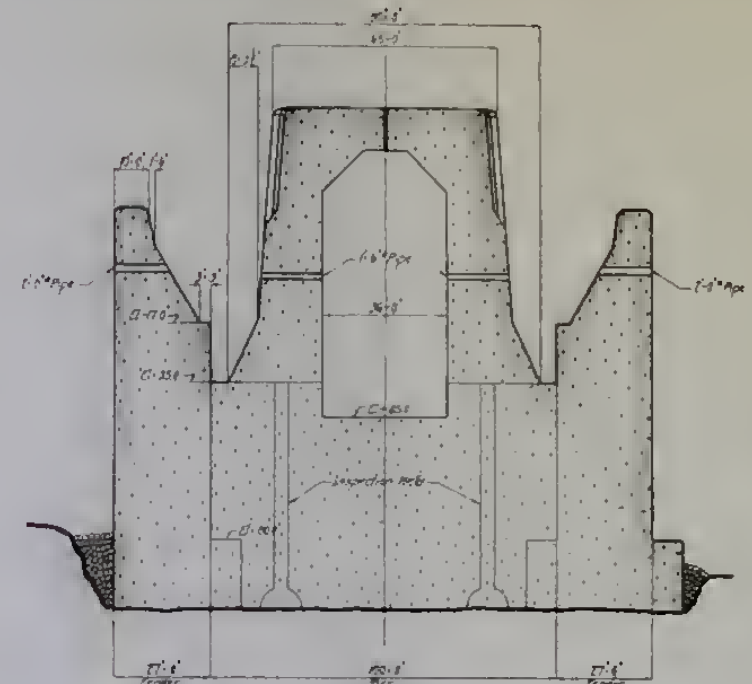
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SCALE IN FEET

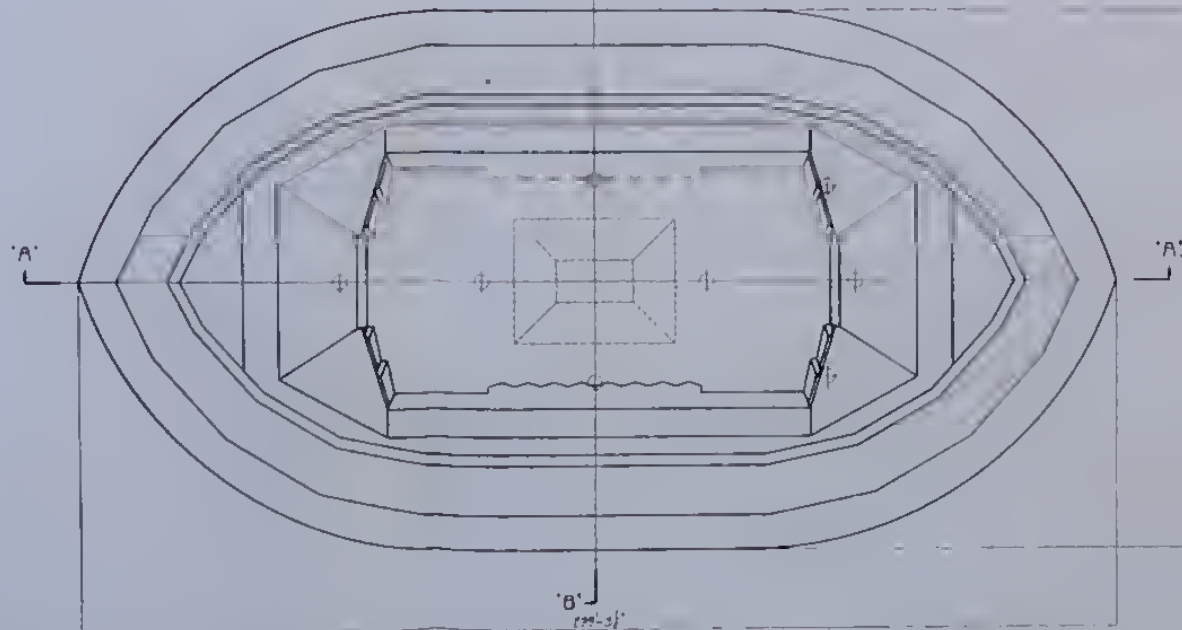




SECTION 'A-A'



SECTION 'B-B'



PLAN

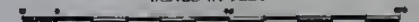
NOTE -
Some concrete for Section A-A is 152' and for Section B-B is 151' and other concrete poured in the dry

THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

**SAN FRANCISCO PIER
GENERAL PLAN AND SECTIONS**

SCALE IN FEET



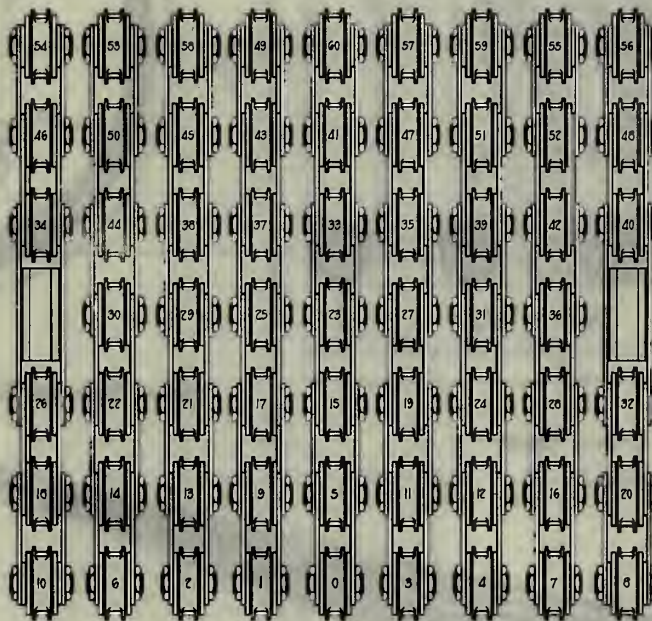
U. S. PUBLIC LIBRARY



2 1/2" Drilled Holes
on Face of 30369.



CABLE STRANDS BEFORE COMPACTING



SECTION 'E-E'

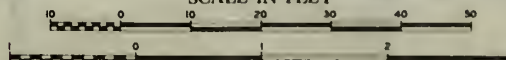
Showing location of strand shoes.

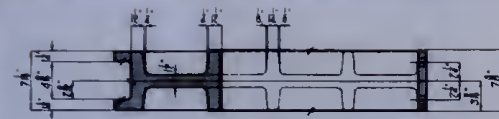
THE GOLDEN GATE BRIDGE SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

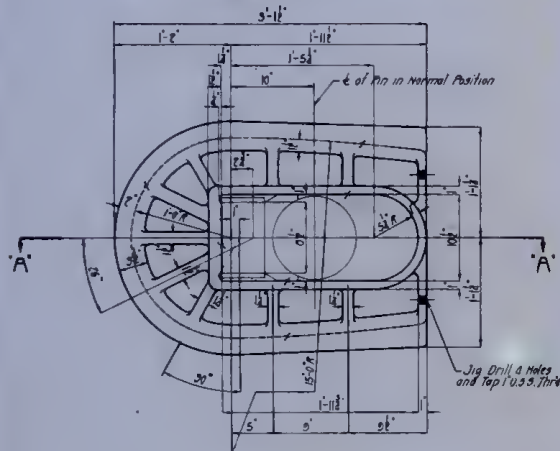
CABLE ANCHORAGE CABLE BAND & STRAND SHOE

SCALE IN FEET

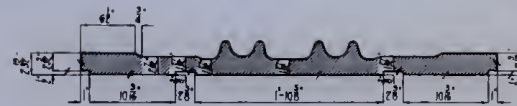




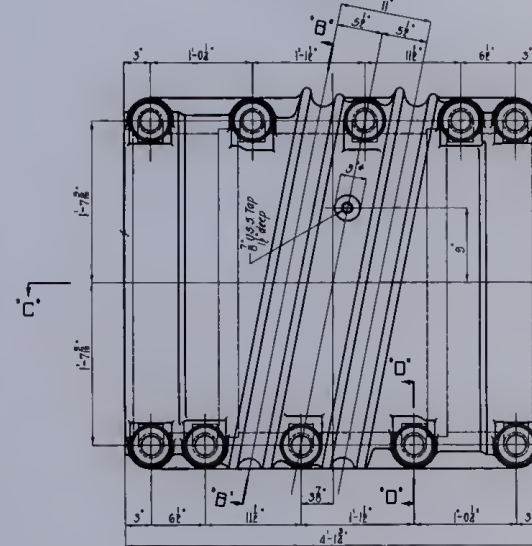
SECTION 'A-A'



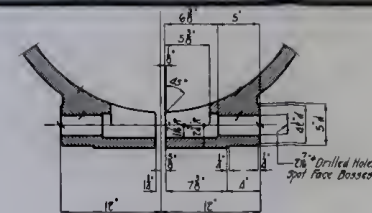
STRAND SHOE



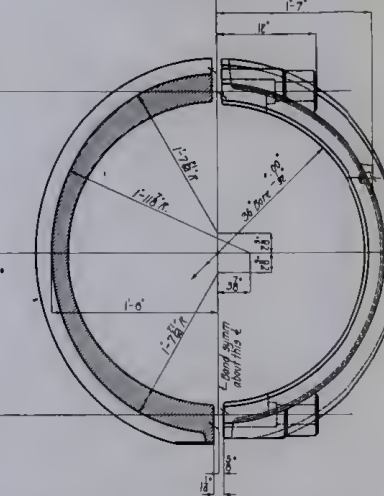
SECTION 'C-C'
Through one half of Band.



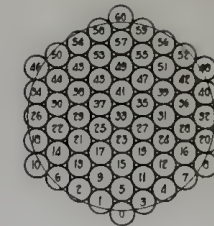
CABLE BAND



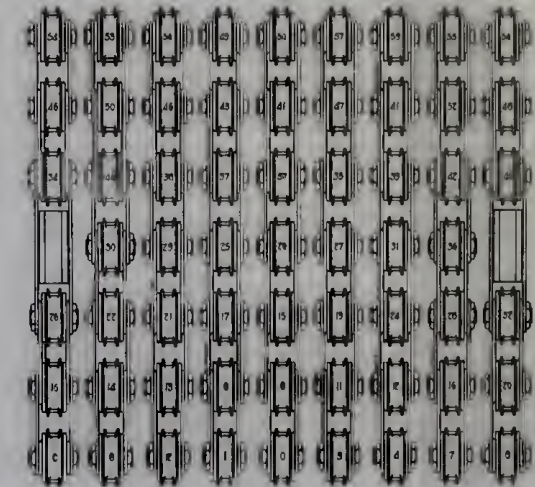
SECTION '0-0'



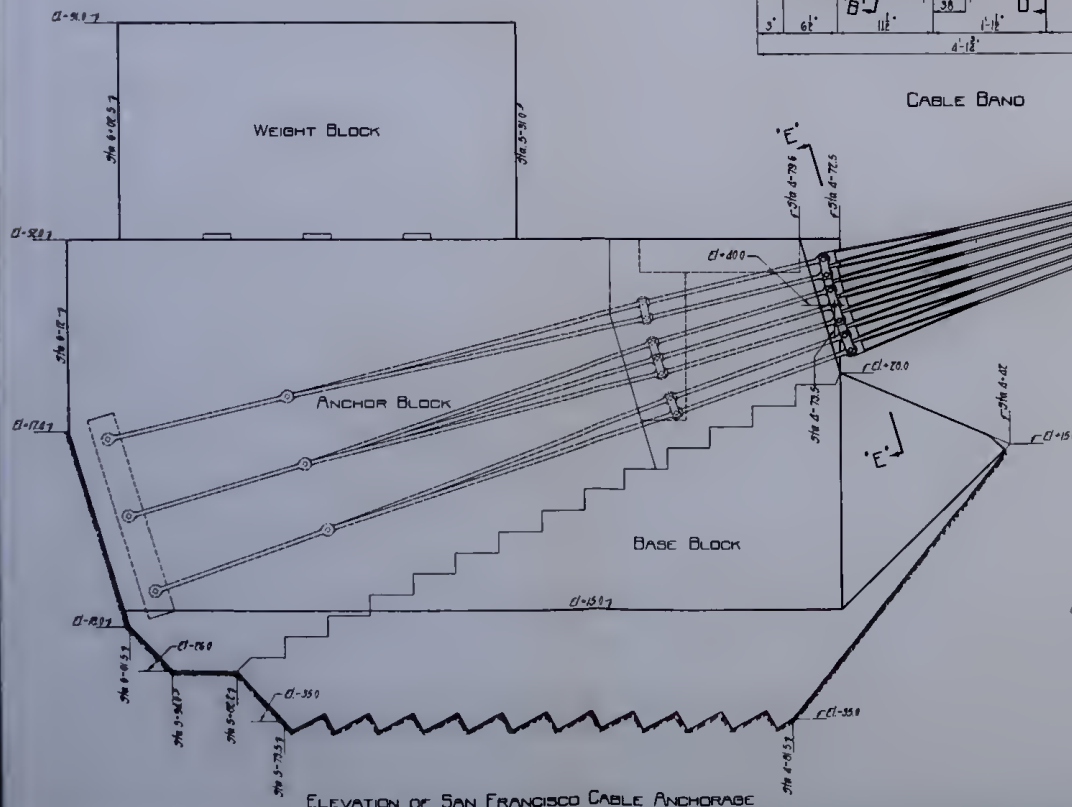
HALF SECTION 'B-B' HALF END VIEW



CABLE STRANDS BEFORE COMPACTING



SECTION 'E-E'



ELEVATION OF SAN FRANCISCO CABLE ANCHORAGE

THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

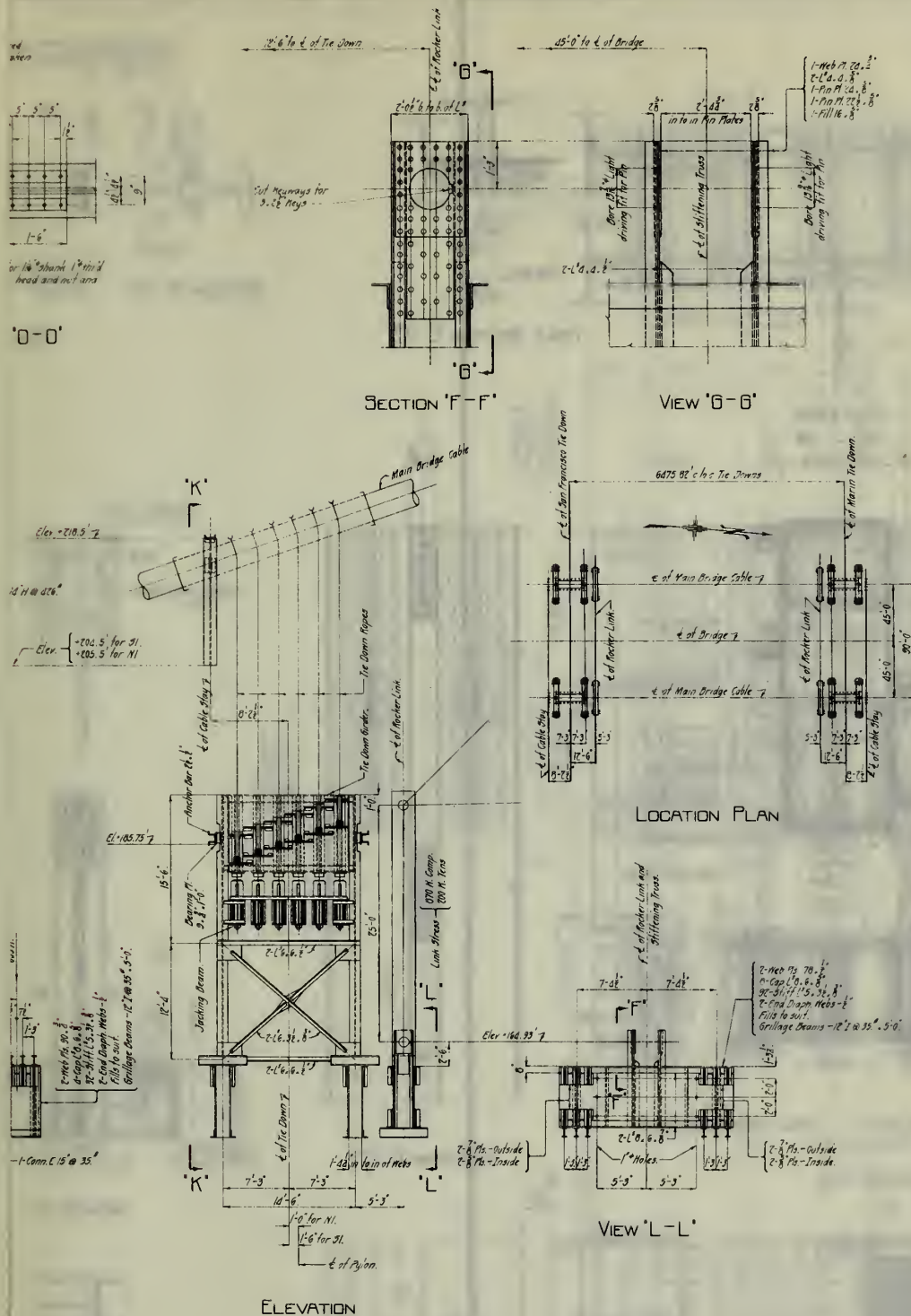
PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

CABLE ANCHORAGE CABLE BAND & STRAND SHOE

SCALE IN FEET





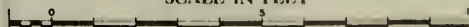


THE GOLDEN GATE BRIDGE SAN FRANCISCO, CAL.

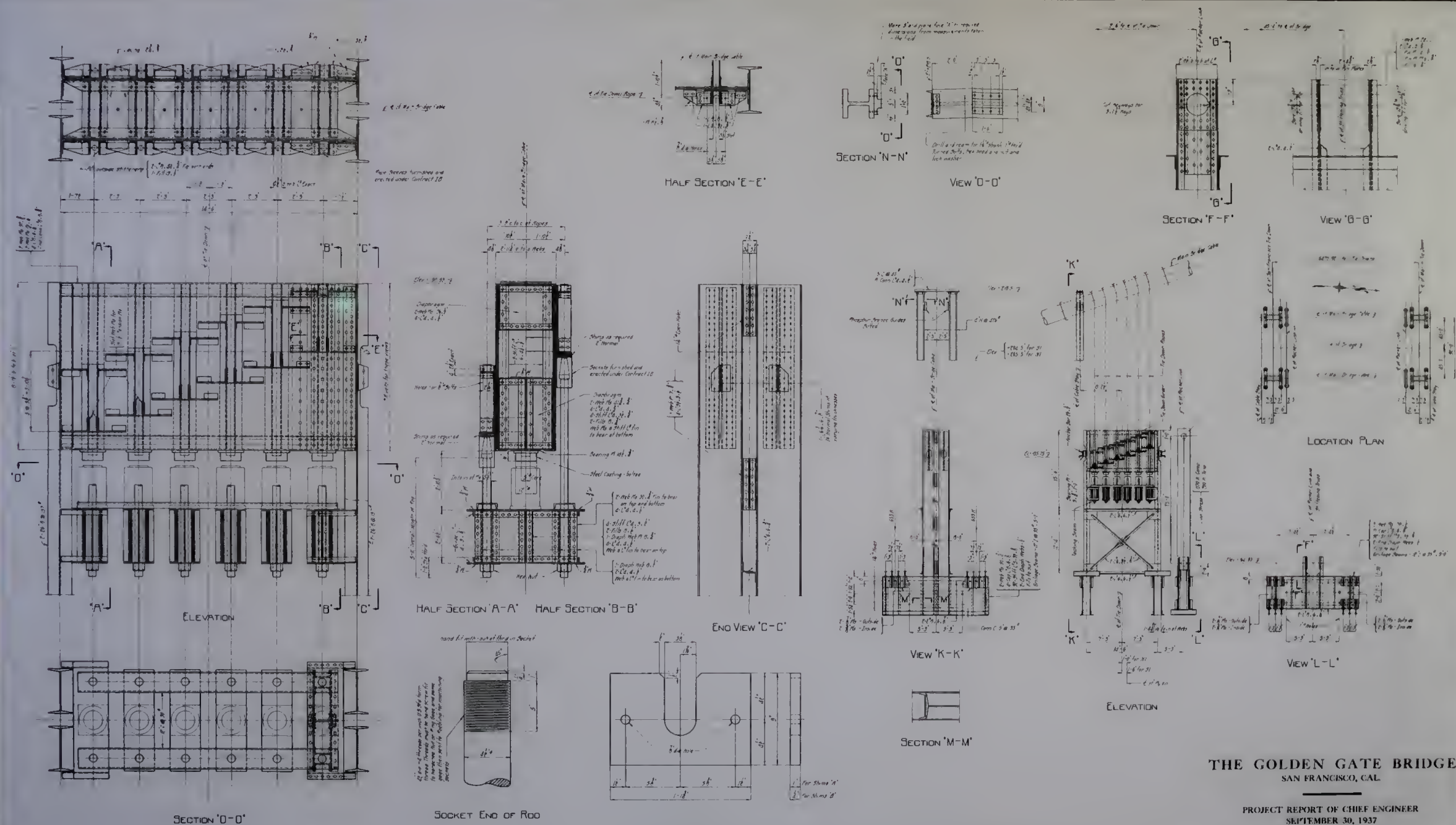
PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

CABLE TIE-DOWN AT PYLONS ASSEMBLY AND DETAILS

SCALE IN FEET



Material Structural Carbon Steel, unless
otherwise noted.
Notes - 1/2", except as noted.
Notes - 1/2", except as noted.



THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

CABLE TIE-DOWN AT PYLONS ASSEMBLY AND DETAILS

SCALE IN FEET

DETAIL OF THE DOWN GIRDER & JACKING BEAM

SHIM PLATE

34 in Males 'A' - 80 Regd
34 in Males 'B' to be finished as regd

GENERAL NOTE -

409.00 Structural Carbon Steel, unless
 otherwise noted
 409.01 1/2" except as noted
 409.02 3/4" except as noted

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D REF ~~740.5 G56~~ 9/05/98

G24.5 G56 Golden Gate Bridge

✓ (5)

Patron discovered plate ~~is~~ torn out.

DRA says SF Hist. Room has 4 copies.

" " L37 has 1 more copy

(2 copies total, including
this document)

SCALE IN FEET

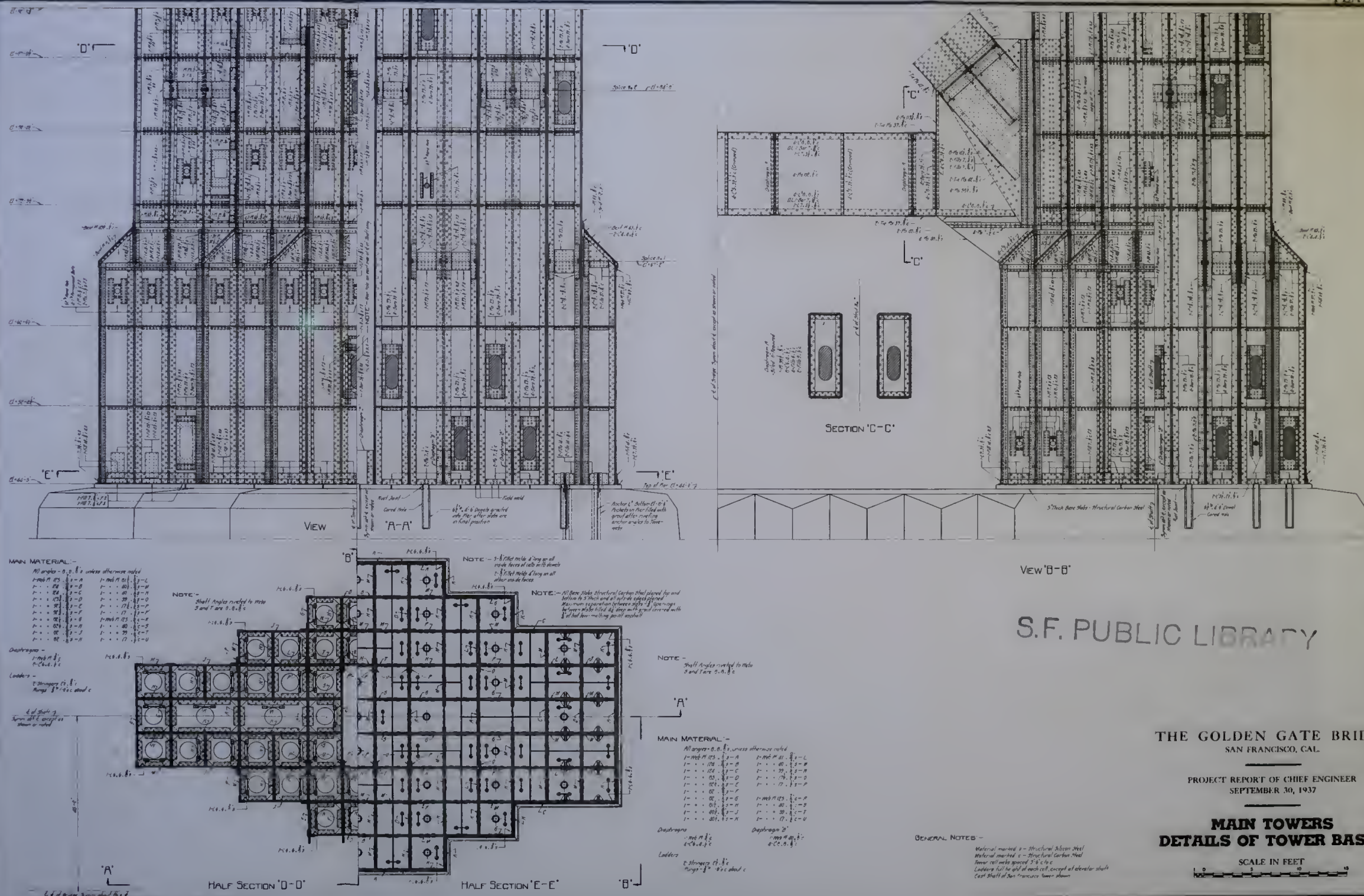


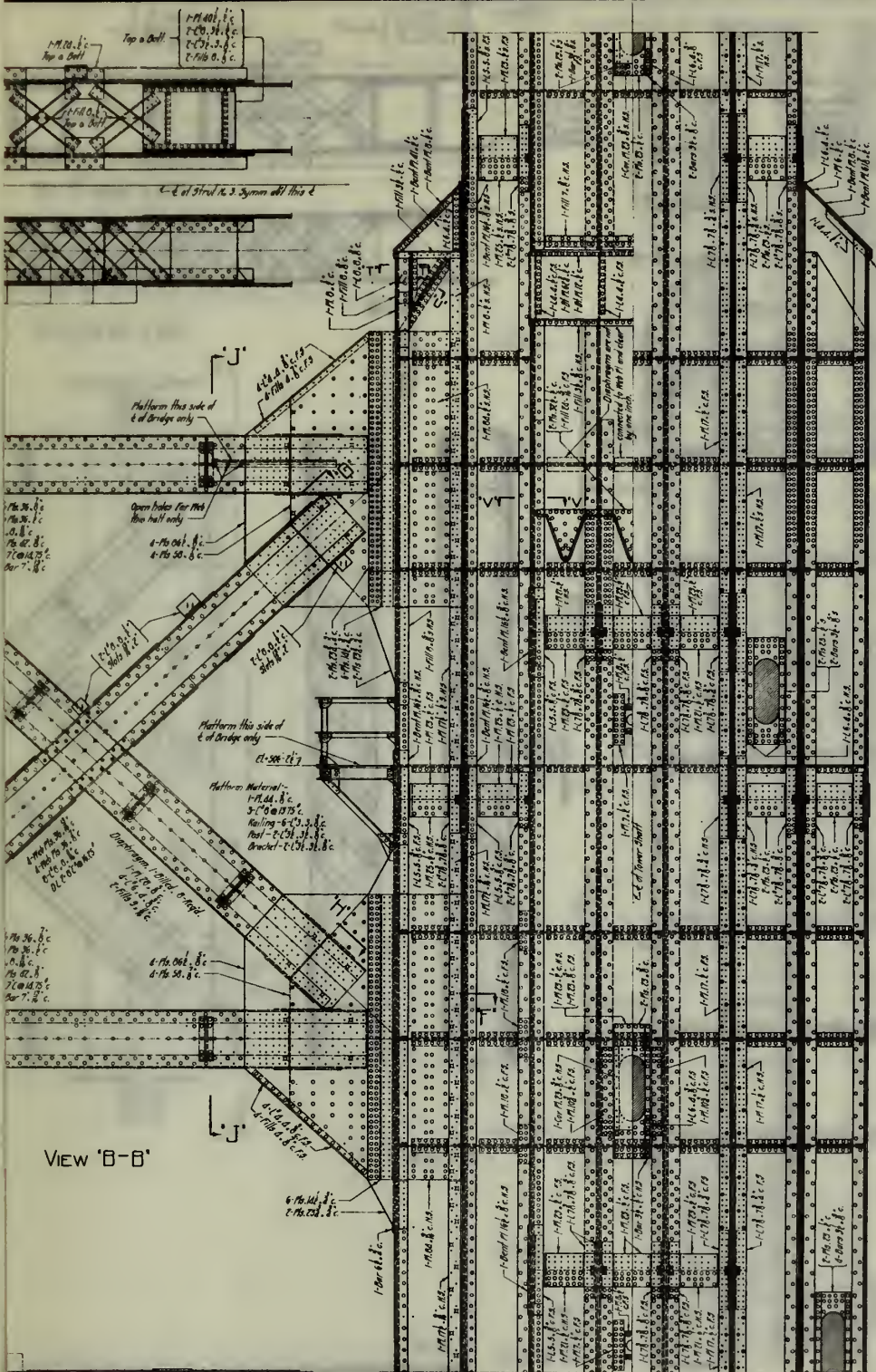
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MAIN TOWERS

DETAILS OF TOWER BASES

Material marked s - Structural Silicon Steel.
Material marked c - Structural Carbon Steel.
Tower cell webs spaced 3'-6" c.to c.
Ladders full height of each cell, except at elevator shaft.
East shaft of San Francisco Tower shown.





THE GOLDEN GATE BRIDGE SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

MAIN TOWERS DETAILS AT STRUT NO. 3

SCALE IN FEET

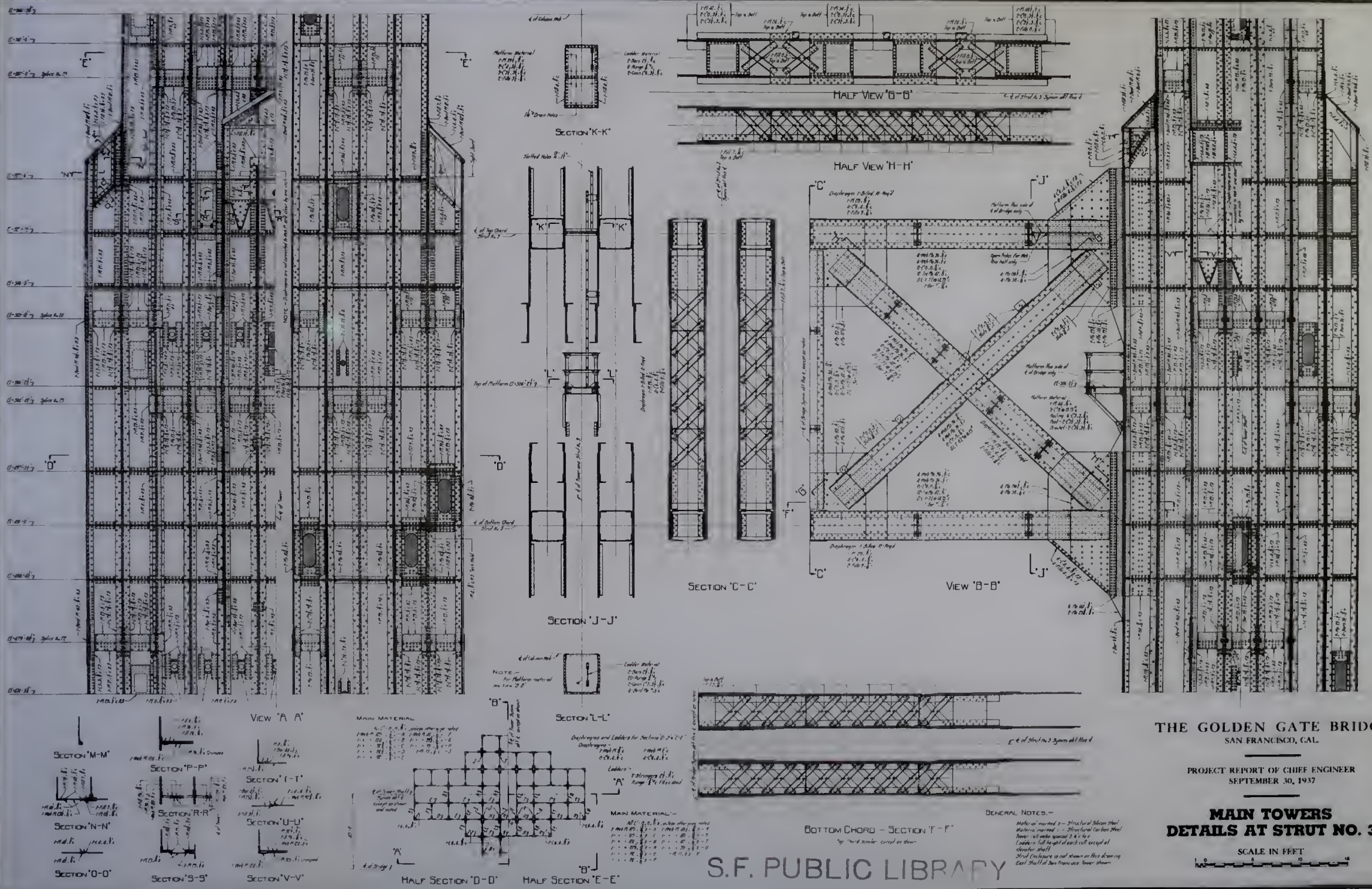


GENERAL NOTES:-

'F-F'

Material marked s - Structural Silicon Steel.
Material marked c - Structural Carbon Steel.
Tower cell webs spaced 5'6" to c
Ladders full height of each cell, except at
elevator shaft
Strut Enclosure is not shown on this drawing
East Shaft of San Francisco Tower shown.

RARY



A horizontal scale bar with markings at 0, 5, and 10.



Handwritten notes on the drawing:

- $\frac{11}{16}$ F. // C.S.
- $\frac{1}{2}$ Cl: p ϕ
- $\frac{11}{16}$ F. // C.S.

SECTION "F-F"



idge

SAN FRANCISCO, CAL.

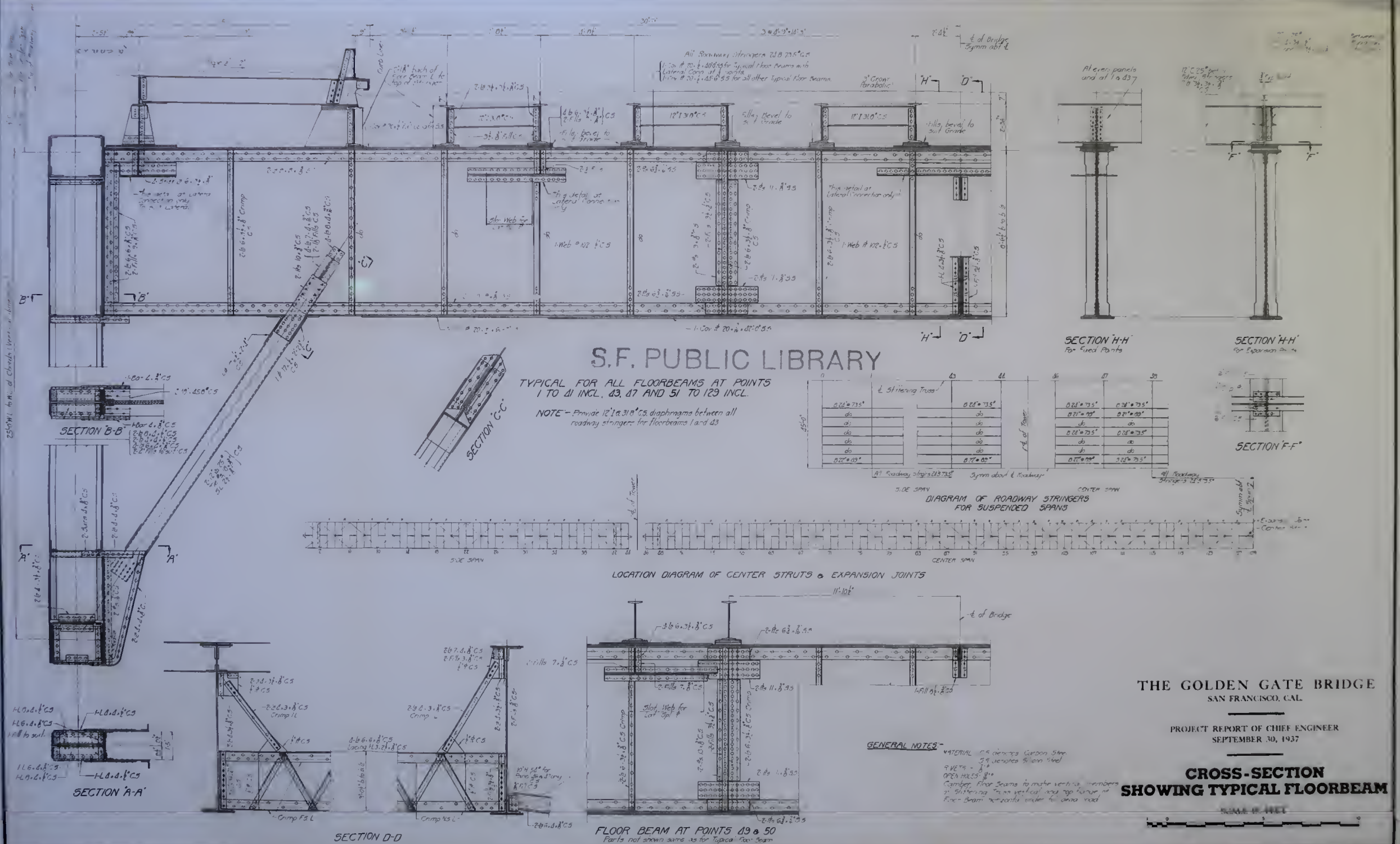
SEPTEMBER 30, 1937

VETS:- 7"φ

**CROSS-SECTION
SHOWING TYPICAL FLOORBEAM**

A scale bar labeled "SCALE IN FEET" with markings for 0, 5, and 10 feet. The bar is divided into segments, with the first segment from 0 to 1 foot being further subdivided into four smaller units.

DMS 12-11-34
 bus 1-5-35
 DMS 1-30-35

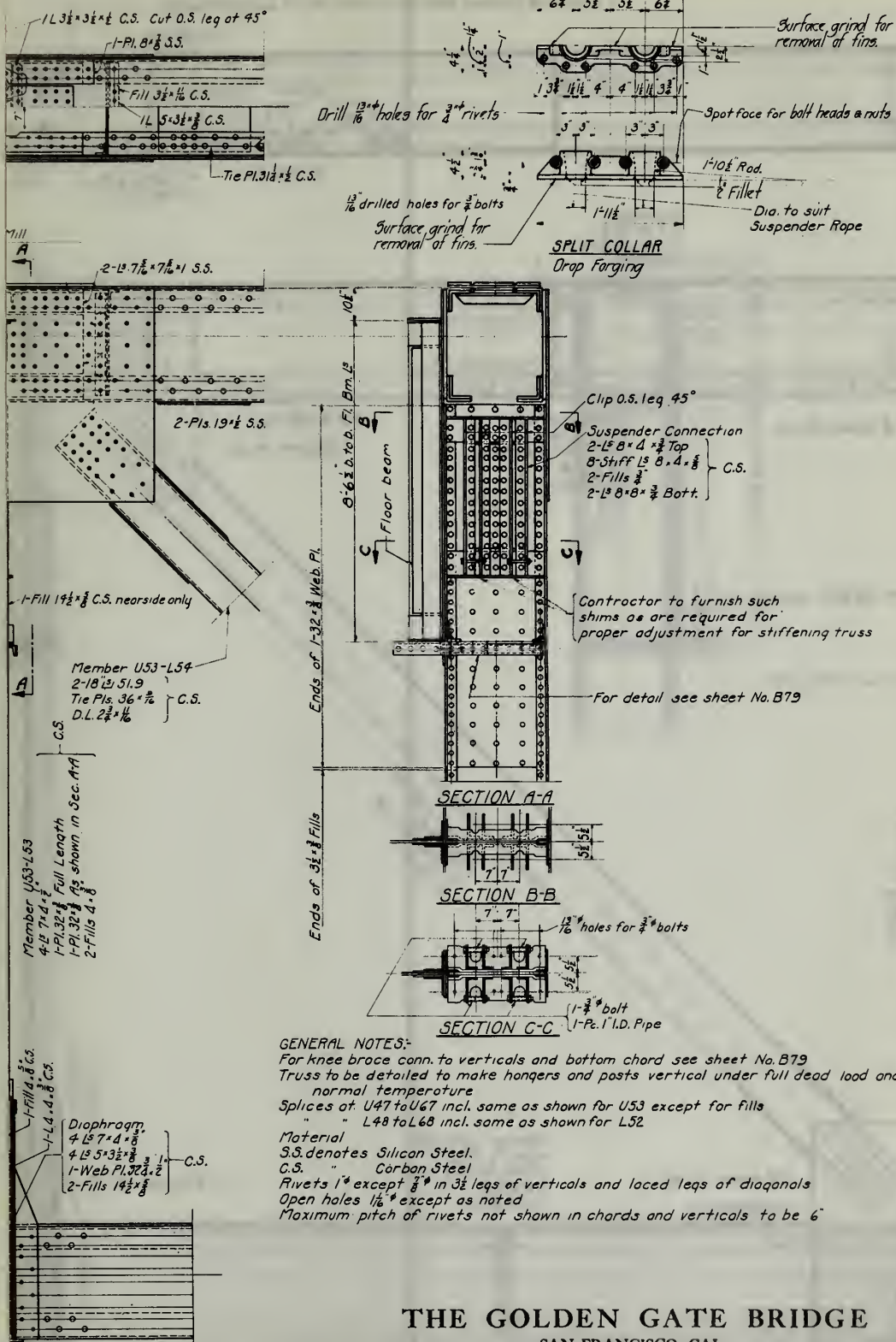


THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

**CROSS-SECTION
SHOWING TYPICAL FLOORBEAM**





THE GOLDEN GATE BRIDGE

SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

STIFFENING TRUSS TYPICAL PANEL OF CENTER SPAN

SCALE IN FEET



Ends of 32" Web Pl.

0'-6" x 10' 0" Fl. Brn Pl.

a

c

Floor beam

Suspenser Connection
 2-L 8" x 8" Top
 8-Stiff L's B.d.
 2-Fills
 2-L 8" x 8" Bott.

C.S.

Contractor to furnish such shims as are required for proper adjustment for stiffening truss

For detail see sheet No. B79

SECTION A-A

1'-0"

1'-6"

1'-0"

SECTION B-B

1'-0"

1'-6"

1'-0"

1/2" holes for 3/4" bolts

1/2" bolt

1" PL I.D. Pipe

SECTION C-C

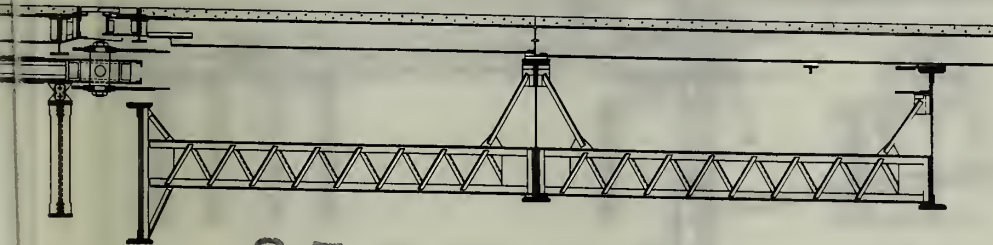
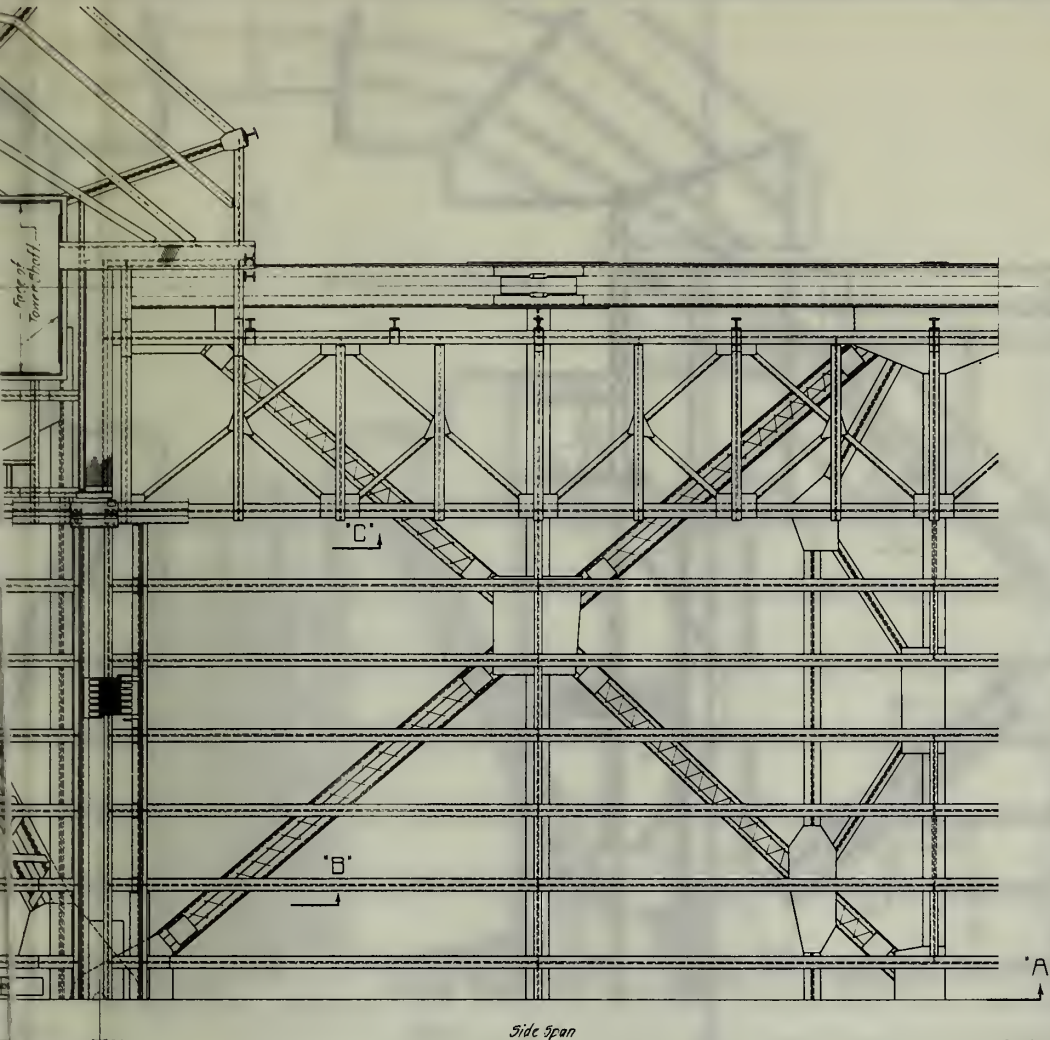
GENERAL NOTES:-
For knee brace connect to verticals and bottom chord see sheet No. 379
Truss to be drilled to make hangers and posts vertical under full dead load and
normal temperature
Splices at U47 to U67 incl. same as shown for U53 except for fills
- L48 to L68 incl. same as shown for L52
Material
S3 denotes Silicon Steel
C5. Carbon Steel
Rivets 1" except 1 1/2" in 3/4" legs of verticals and laced legs of diagonals
Open holes 1 1/4" except as noted
Maximum pitch of rivets not shown in chords and verticals to be 6"

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

**STIFFENING TRUSS
TYPICAL PANEL OF CENTER SPAN**

A horizontal scale bar labeled "SCALE IN FEET" at the top. The bar is marked with numbers 0, 5, and 10. The segment between 0 and 5 is divided into four equal parts, each representing 1.25 feet. The segment between 5 and 10 is also divided into four equal parts, each representing 1.25 feet. The total length of the scale is 10 feet.

1-5-35
 1-3-35
 1-2-35
 1-1-35
 1-1-35



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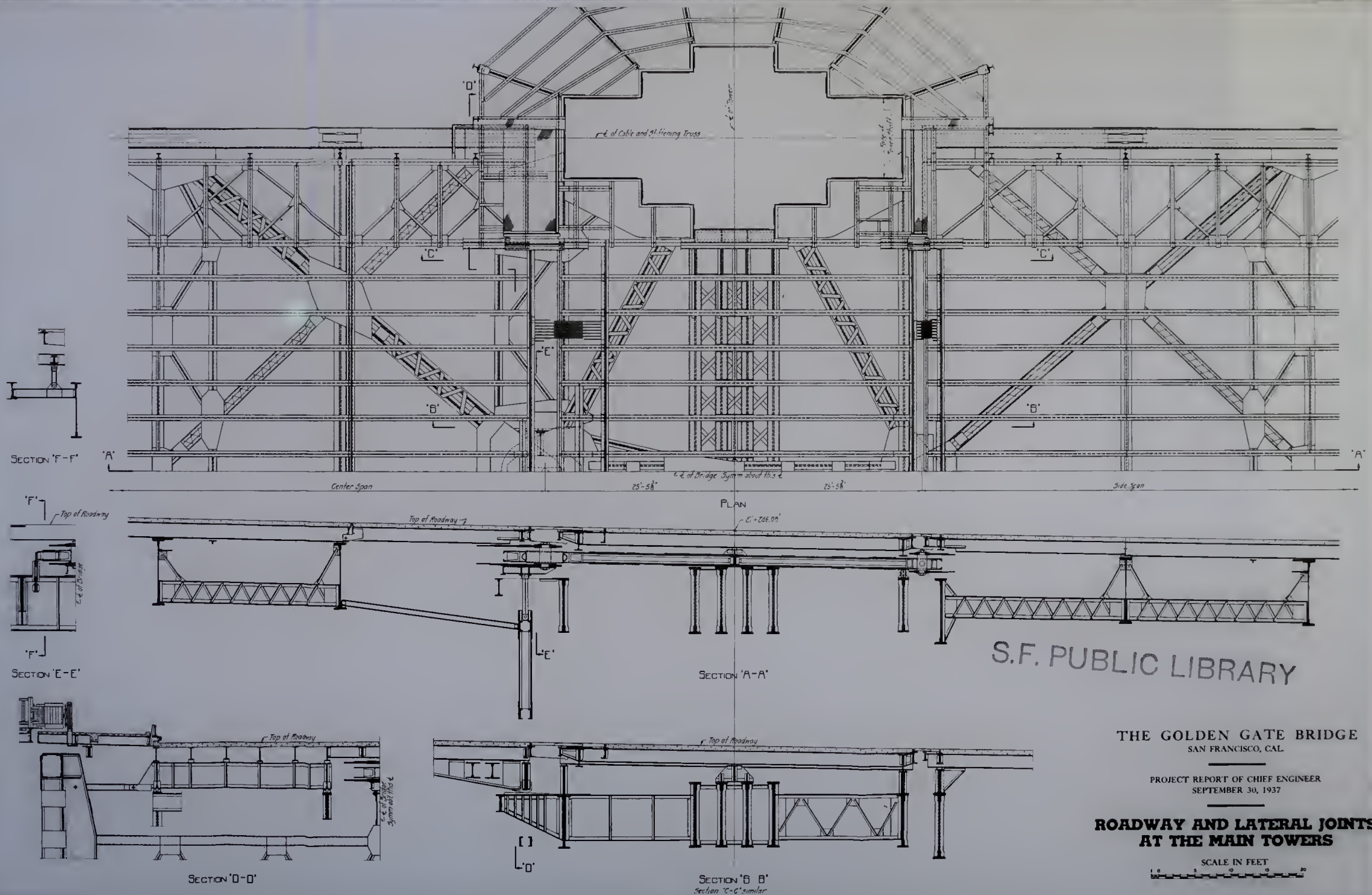
THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

**ROADWAY AND LATERAL JOINTS
AT THE MAIN TOWERS**

SCALE IN FEET

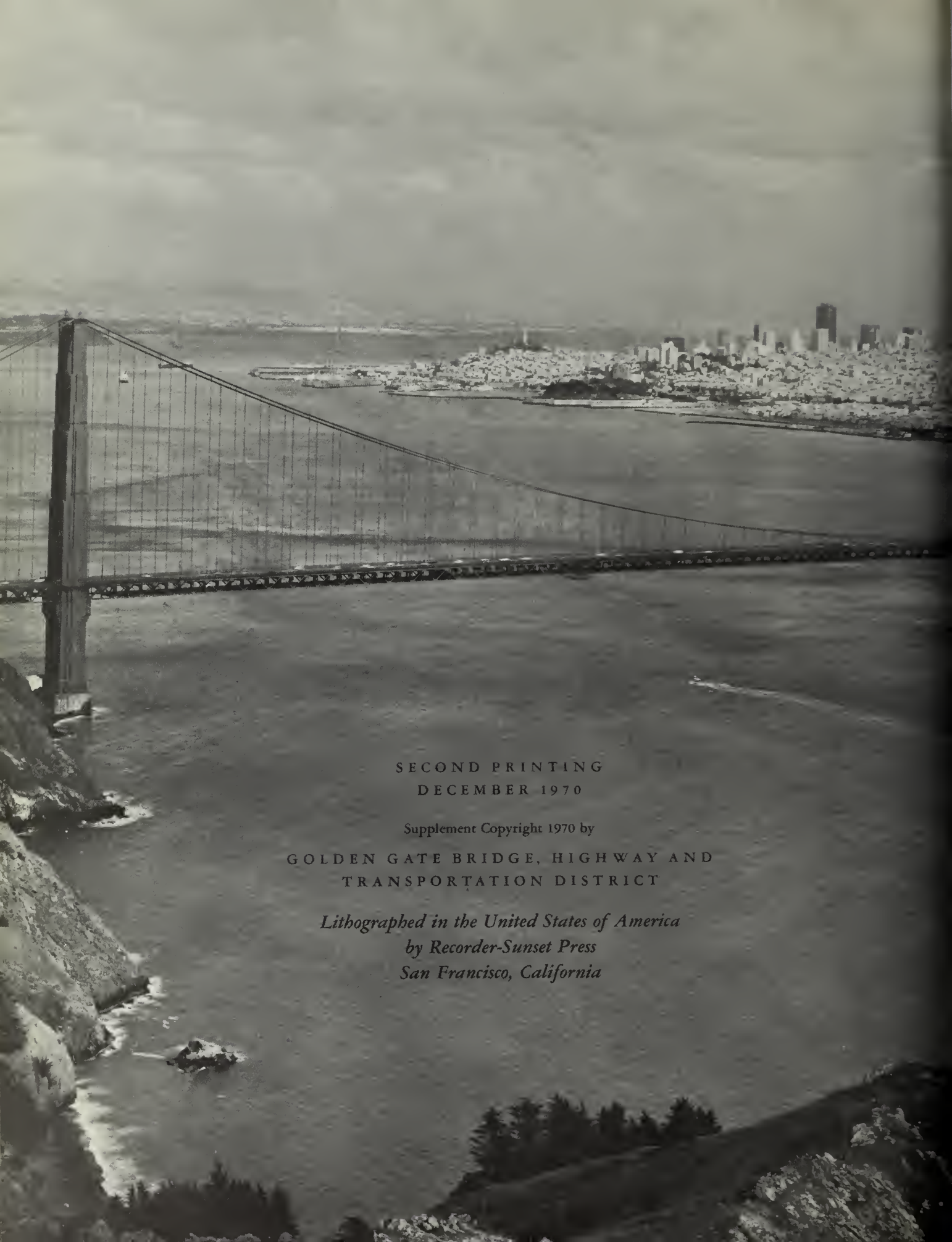






**SUPPLEMENT TO THE
FINAL REPORT OF THE CHIEF ENGINEER
SEPTEMBER, 1937**

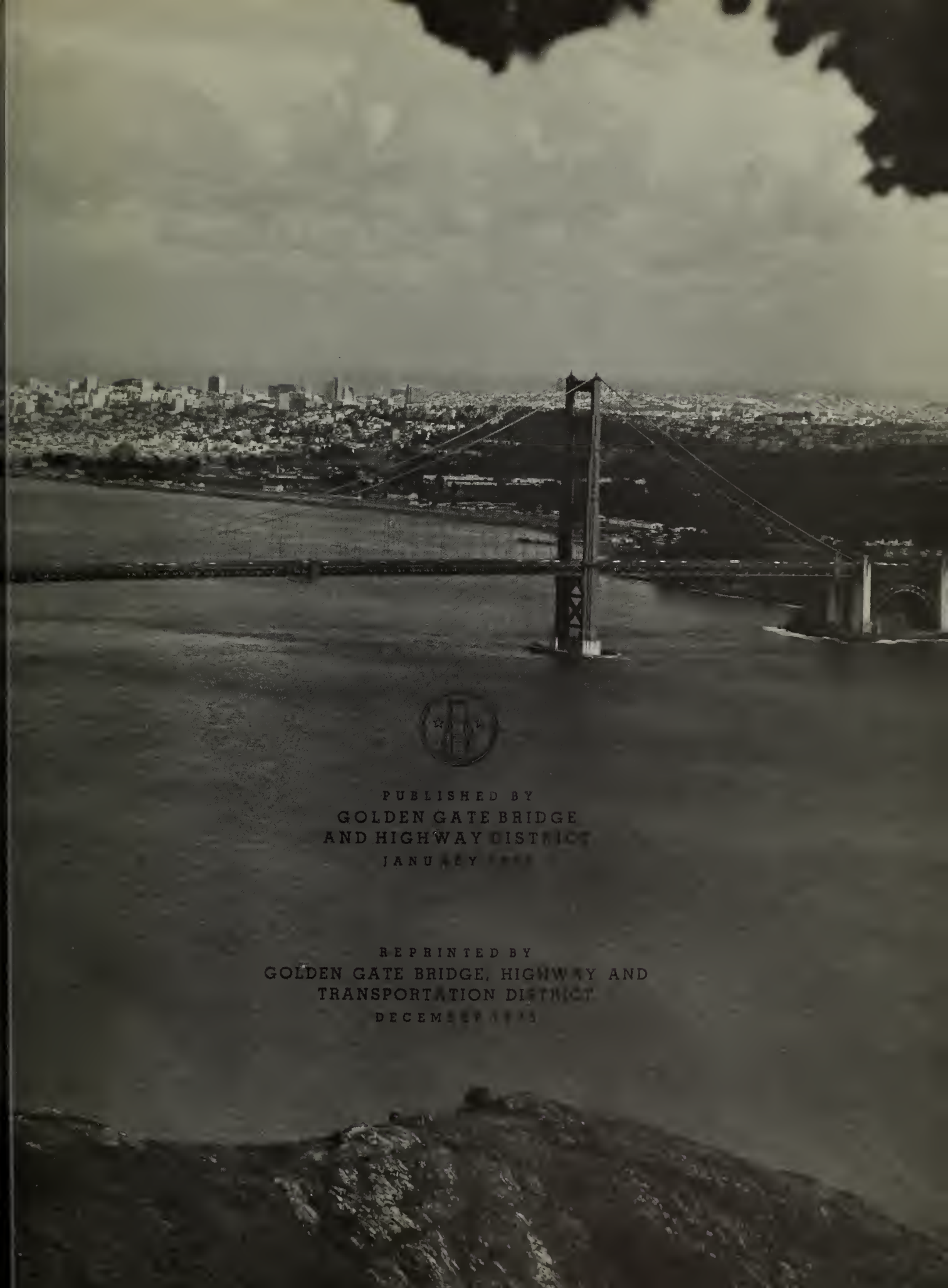
**By
CLIFFORDE. PAINE**



SECOND PRINTING
DECEMBER 1970

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San Francisco, California*



PUBLISHED BY
GOLDEN GATE BRIDGE
AND HIGHWAY DISTRICT
JANUARY 1933

REPRINTED BY
GOLDEN GATE BRIDGE, HIGHWAY AND
TRANSPORTATION DISTRICT
DECEMBER 1973

FORWARD

This edition of the "Report of the Chief Engineer to the Board of Directors of the Golden Gate Bridge and Highway District" has duplicated in every detail the original work written by Joseph Strauss upon completion of the Golden Gate Bridge and published in January, 1938. One third of a century later, the District is reprinting the Report so that public libraries, technical reference works, consultants, and others with a need to know the District's history and highlights of the span's structural design may have copies available. The 5000 copies originally printed have been depleted for many years, and originals are virtually unobtainable.

In addition to the Chief Engineer's Report, this edition contains supplementary information on modifications to the structure since opening day. On December 1, 1951, unusual and severe winds caused the bridge to be closed to traffic. Subsequent inspection and analysis performed under the direction of Clifford E. Paine, Consulting Engineer, resulted in the major structural modifications designated as the Bottom Lateral Bracing System.

Mr. Paine was Principal Engineer with Joseph Strauss during the bridge's design and construction, and has been closely associated with the Bridge District during the intervening years. He was retained by the District to document these changes in the Supplement which is bound within. His position and experience make him pre-eminently qualified to do so.

For the Board of Directors,

William Moskovitz
President

December 1, 1970

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INTRODUCTION

Since the Golden Gate Bridge was completed and opened to traffic in 1937 there have been very few important changes in or additions to the structure. It is the purpose of this supplement to point out and discuss these changes and additions.

The failure of the Tacoma Narrows Bridge in 1940 focused attention on certain important structural requirements of a cable suspension bridge beyond its ability (1) to carry live load on its roadway and (2) to withstand static wind loads impressed on its exposed surfaces.

Wind storms of note occurred on February 9, 1938; on February 11, 1941; and on December 1, 1951.

A Bottom Lateral System was added in 1954 in all suspended spans.

Traveling Scaffolds were added in 1955. One set of scaffolds was placed in each side span and two in the main span.



GENERAL HISTORY

FAILURE OF THE TACOMA NARROWS BRIDGE IN 1940. The cable suspension bridge over the Tacoma Narrows which was opened to traffic on July 1, 1940 and failed on November 7, 1940, comprised a central span of 2800 ft. and two side spans of 1100 ft. each. Its two cables, each about 17 inches in diameter and spaced 39 ft. center to center, passed over tower tops 450 ft. above low water.

The suspended structure with stiffening girders 8 ft. deep carried a reinforced concrete roadway 26 ft. wide and two 5 ft. sidewalks resting directly upon five lines of steel stringers framed into transverse floorbeams which were spaced 25 ft. center to center. The floorbeams were framed into the stiffening girders which were 39 ft. apart in the planes of the two main cables from which they were suspended.

A lateral bracing system was provided in a plane about 2 ft. above the bottoms of the stiffening girders.

The suspended structure described above may be classified as one having cross-section with two sides and bottom closed leaving the top open. The construction could not have accommodated a second lateral frame. For maximum torsional resistance the cross-section should be closed on all four sides as is obtained by employing two stiffening trusses, one on each side, and two lateral systems, one in the plane of the top chords and one in the plane of the bottom chords.

Immediately after being put into service the Tacoma Bridge frequently developed undulations of the roadway so severe as to cause discomfort to the users of the bridge. Vertical oscillations having waves with double amplitudes of five feet were not uncommon. Sometimes wind velocities of four miles per hour would produce pronounced waves while at other times no waves were produced at wind velocities of thirty miles per hour.

On the morning of November 7, 1940 a southerly wind with velocity recorded at forty-two miles per hour was striking the bridge at a quartering angle. The wind had continued for several hours prior to 9:30 A.M. at which time the main span was vibrating with a frequency of 36 cycles per minute and a double amplitude of about three feet. The roadway was not twisting at this time. The cables were vibrating in phase with one another.

Shortly after 10:00 A.M. there was a change in the mode of vibration at which time the main span vibrated in two segments with a node at mid-span and a frequency of 14 cycles per minute. Then the cables began vibrating out of phase with one another resulting in a twisting motion of the roadway, one side going down as the other side came up.

A little later the frequency changed from 14 to 12 cycles per minute. Motion pictures show a tilting of the bridge deck amounting to more than 30 degrees each way

from the horizontal. The double amplitude of the waves probably attained a maximum of 28 feet. The main span began to break up just before 11:00 A.M.

The Tacoma Narrows Bridge, built at a cost of \$6,469,770.00, was insured for \$5,200,000.00. This insurance was divided among twenty-two insurance companies.

The Narrows Bridge Loss Committee representing the twenty-two original insurers of the bridge, on November 9, 1940 engaged Clifford E. Paine, Consulting Engineer, Chicago, Ill. to investigate and report on the cause of the failure and the extent of the loss.

In February, 1941, at Mr. Paine's suggestion, four other engineers were named to advise and collaborate with him and to comprise with him a Board of Consulting Engineers. The Engineers thus named were as follows:

Hardy Cross, Professor of Civil Engineering at Yale University; Shortridge Hardesty, Consulting Engineer of New York City; Holton D. Robinson, Consulting Engineer of New York City; Wilbur M. Wilson, Research Professor of Structural Engineering at the University of Illinois.

Mr. John M. Carmody, Administrator, Federal Works Agency, appointed a Board of Engineers to investigate and report on the failure. This Board consisted of Othmar H. Ammann, Consulting Engineer of New York City; Theodore von Karman, Director of the Daniel Guggenheim Aeronautical Laboratory at the California Institute of Technology, Pasadena, California; and Glenn B. Woodruff, Consulting Engineer of San Francisco, California. The report of this Board entitled "The Failure of Tacoma Narrows Bridge" is dated March 28, 1941.

A few days after his engagement by the Loss Committee the writer was joined in Seattle by his assistant, Mr. Charles H. Clarahan, Jr. The two worked together making analytical studies of the ill-fated span's construction and its behavior. The objective of this phase of the study being (1) to find its weaknesses and (2) to find how the behavior of the span could be corrected by changing its design. (The scope of their investigation was much broader than is here indicated but the other phases of the study are not included here because they would not contribute to the purpose for which this discussion is included in the Supplement.)

One did not have to look far to find several examples of cable suspension bridges with suspended spans similar in construction to the Tacoma Bridge having plate girder stiffening members with a single plane of lateral bracing and having a history of unsatisfactory behavior. Most had vertical oscillations of such magnitude as to require corrective treatment. Some had torsional oscillations in which the deck tilted alarmingly. The analytical studies placed the Tacoma Narrows Bridge in the lead so far as this misbehavior was concerned.

It was concluded that the suspended structure of the Tacoma Bridge could have been improved in design by modifying it as follows:

1. Employ trusses instead of plate girders for the stiffening members.
2. Put a top lateral system in the plane of the top chords and a bottom lateral system in

the plane of the bottom chords.

3. In conjunction with the above have cross-frames at intervals to maintain a rectangular cross-section.

A design which embodied the above changes was developed and for convenience in reference was named "The Tacoma Fully Braced Design."

The report to the Narrows Bridge Loss Committee states "The comparative effectiveness of suspension bridge designs to resist irregular and turbulent wind forces is reflected by their torsional deformations under assumed asymmetric loads and torsion. The desired behavior is attained when such loads cause comparatively little twisting of the suspended structure.

In their report on the failure of the Tacoma Narrows Bridge, Messrs. Ammann, von Karman and Woodruff made comparison of the twisting or tilting of the bridge floors of several designs under two different conditions of loading." The comparison which is to be discussed herein is based on one of these, namely on the tilting of the floor resulting from an asymmetric load of 100 lbs. per ft. applied on the near cable from the tower to mid-span and a similar load applied on the far cable from mid-span to the other tower.

They made several modifications of design of the Tacoma Narrows Bridge to compare how the various changes in design should affect their behavior. The results in terms of tilting were reported as follows:

1. Bridge as built except unstiffened. Tilting is over 5%.
2. Bridge as built. Tilting is a little over 5%.
3. With weight increased 10%. Tilting about 4.6%.
4. With 16 ft. stiffening trusses. Tilting about 4.2%.
5. With 24 ft. stiffening trusses. Tilting about 3.8%.
6. With 32 ft. stiffening trusses. Tilting about 3.3%.
7. With 100% increase in weight and cables spread to 53 ft. Tilting about 2.0%.
8. With 100% increase in weight, 24 ft. stiffening trusses and cables spread to 53 ft. Tilting about 1.8%.

Looking over the results of the eight modifications of the Tacoma design as reported by Messrs. Ammann, von Karman and Woodruff it is seen that the minimum tilting they attained for the asymmetric loading assumed was in design modification No. 8



Joseph B. Strauss, Chief Engineer (left), and Clifford E. Paine, Principal Engineer, are shown on a tour of inspection of the bridge as it is nearing completion in this 1937 photograph.

in which the weight of the bridge was increased 100%, stiffening trusses 24 ft. deep added and the cables were spread to 53 ft. center to center. This yielded a tilting of 1.8%.

Studies made by Mr. Claraham and the writer showed that the most effective means of obtaining torsional stiffness in the suspended structure was to use stiffening trusses with top laterals between the top chords and bottom laterals between the bottom chords. This produces a "closed" cross-section, that is to say all four sides of the cross-section are braced. This construction was previously referred to as the "Tacoma Fully Braced Design." The tilt of this design with stiffening trusses 25 ft. deep and with lateral bracing top and bottom, with no change in weight or change in spacing center to center of cables, was about 0.4%. This is less than one quarter of the tilt obtained with design No. 8 described above.

It follows that any cable suspension bridge having a suspended structure with cross-section closed on two sides by stiffening trusses, on top by deck and top lateral bracing but open on the bottom can obtain a great increase in torsional stiffness by the addition of a bottom lateral system in conjunction with effective cross-frames at intervals sufficient to maintain a rectangular cross-section.

The torsional stiffness of the suspended structure in the "Fully Braced Design" was sufficient to force the cable on the unloaded side to deflect downward at the quarter point. Hence a very substantial part of the load on the load cable was transferred to the unloaded cable.

It should be noted that when a cable suspension bridge is vibrating all parts of the structure including the towers participate in the motion to some degree. During part of the cycle some energy of motion (kinetic energy) is built up and transformed into energy of position (potential energy). In the other part of the cycle the potential energy is converted back to kinetic energy. Some energy is lost through structural damping and windage but also sometimes the wind continues to feed energy into the vibrating structure. As the amplitude of the waves increases so also does the structural damping per cycle. Generally structural damping in trusses exceeds that in plate girders. The strain in steel members produces heat which is one form of energy lost in every cycle of vibration. Slippage at joints adds further to the loss of energy and so contributes to structural damping.

EARLY STORMS. The first severe wind at the Golden Gate Bridge occurred on February 9, 1938. Westerly winds reached velocities which made it impossible for a man to stand erect on the sidewalk. No instrumental observations were made of the movements of the bridge during the storm, but witnesses reported undulations of appreciable amplitude.

The next severe storm came on February 11, 1941 when a fairly high wind from the southwest struck at an angle of about 45 degrees with the axis of the bridge. The longitudinal axis of the bridge bears S 5°-26' E. The wind continued for over three hours with gusts reaching an intensity of 60 miles per hour. It is reported that the bridge deflected laterally 4.9 feet and that the maximum vertical movement at the quarter

point of the main span was 2 feet and the frequency $7\frac{1}{2}$ cycles per minute. The report made no mention of torsional movements.

INSTRUMENTAL INSTALLATIONS AND OBSERVATIONS. In 1942 a recording anemometer was installed at the center of the main span. This instrument records the velocity and horizontal angle of the wind.

A target was installed at mid-span so that lateral movement of the roadway could be read directly with the aid of a transit set up at the San Francisco Tower. At this time also there was installed on the sidewalk at the center of the main span an accelerometer to measure the vertical oscillations at that point.

Additional accelerometers were installed in 1945 and 1946, so that from that time until after the storm of December 1, 1951 there is a continuous record of wind velocities and directions at the center of the main span and of vertical movements of the



bridge floor at the ten stations where accelerometers were installed. The stations are located on both sides of the roadway at the centers of all three spans and at the south quarter point of the main span. Instruments are also stationed on the west side of the roadway only at the three-eighths point and at the north quarter point of the main span.

The drum speed on these recording accelerometers has proved to be too slow. As a result the graph recording the vibrations is not spread out sufficiently to show clearly the individual waves when the amplitudes of the motion are large and overlapping.

A study of the vibration charts together with the wind charts recorded over the six years following the installation of the instruments leads to some general conclusions as follows:



The box-like suspended structure comprises a top provided by the floor deck, two vertical sides provided by the stiffening trusses and a bottom provided by the Bottom Lateral System. The vertical hangers tying the transverse struts to the floor beams above, combined with the kneebraces of the original construction, all lying in vertical planes comprise an effective cross-frame at each panel point. The main requisites for good torsional resistance have thus been met.

South winds as high as 30 to 50 miles per hour and of several hours duration are not uncommon. These winds cause little, if any, motion to the bridge. For example on February 6, 1949 there was a south wind averaging 30 to 40 miles per hour for a period of six hours. Vibrations reached a double amplitude of only 11 inches. The highest double amplitude recorded with a south wind was 14 inches on January 16, 1950. This was built up during a period of eight hours with wind velocities averaging 30 to 40 miles per hour and gusts reaching 40 to 50 miles per hour.

No considerable motion has been recorded by winds making an angle of less than 45 degrees with the longitudinal axis of the bridge.

Winds over 30 miles per hour from an easterly quarter making an angle of more than 45 degrees with the longitudinal axis of the bridge are rarely recorded and none has caused any noteworthy motion.

The greatest movement recorded prior to the December 1, 1951, storm occurred on June 6, 1950 when a double amplitude of 45 inches was attained at the middle of the main span. The wind was from the west with a peak velocity of 56 miles per hour. The main span vibrated as a single segment at a frequency of 8 cycles per minute. For brief periods this motion combined with torsional vibrations of the first asymmetric mode (double segment).

THE STORM OF DECEMBER 1, 1951. On December 1, 1951 a strong southwest wind, which had persisted through the early part of the day, increased in intensity until at 3:05 P.M. it reached a velocity of 50 miles per hour with frequent gusts of 55 miles per hour. From 4:30 to 5:30 the velocity in general remained at about 50 miles per hour with gusts up to 64 miles per hour. For a period of twenty minutes, 5:40 to 6:00, the velocity held at an average of 55 miles per hour with gusts of 60 to 69 miles per hour. The peak of 69 miles per hour came at 5:55. From that time on the wind velocity steadily diminished so that at 7:00 it was reduced to a little more than 40 miles per hour with gusts of 50 miles per hour or less. At 9:00 the velocity ranged between 30 and 45 miles per hour.

The wind direction at 3:00 was southwest. At 3:30 it became $22\frac{1}{2}$ degrees south of west and held that direction fairly constantly until 7:00 when it gradually shifted westward. From 8:00 to 9:00 it was west.

Accelerometer charts show the usual pattern of vibrations during the early period of the storm. Random vibrations in the main span gradually built up into a well defined motion at 3:00, at which time the main span was vibrating in the first symmetric vertical mode (single segment) with a frequency of about 8 cycles per minute. At 5:30 the frequency at mid-span had increased to 10 cycles per minute and at 5:45 to 16.8 cycles. Possibly a coupling of the first asymmetric torsional mode with the first symmetric vertical mode was developing during this period. These mid-span oscillations attained a double amplitude of 22 inches.

At 5:55, at the peak of the storm a double amplitude of about 130 inches is indicated at the southeast quarter point of the main span. Due to mechanical trouble the chart from the instrument at this station is not decisive beyond this time. The instrument stationed at the southwest quarter point registered a double amplitude of 108 inches.

At 6:10 the average wind velocity had dropped to 50 miles per hour with occasional gusts of 60 miles per hour. At that velocity the amplitude decayed rapidly. The marked reduction started at 6:12 or two minutes after the average wind velocity had dropped to 50 miles per hour. By 6:30 the amplitude was reduced 50% and by 7:00 it was on the order of 20 inches. During the time from 5:15 to 6:12 the double amplitude had built up from 20 inches to the maximum of about 130 inches. Decay started as soon as the average wind velocity dropped to 50 miles per hour.

Because of the slow speed of the chart the graph is too compact. This fact, coupled with overlapping of graphs, makes it impossible to determine precisely the phase relationship between the east and west cables. A comparison of charts from the southeast and southwest quarter points indicates that the cables were not always moving in phase with one another and that the difference varied from nothing up to one half cycle.

The floor structure of the main span is freely suspended from the cables. Its ends are held against lateral movement at the towers. At normal temperature free longitudinal motion is provided to the extent of 18 inches in either direction. During the height of the storm the suspended structure of the main span oscillated longitudinally



Public safety and convenience required that as much materials handling and erection as possible be done out of traffic and out of sight of the motorist. The Bottom Lateral Bracing System contract involved approximately \$3,500,000, almost 5,000 tons of structural steel, and a year and a half of time.

through the full length of travel provided. This motion possibly had something to do with the changing phase relationship of the two cables.

On the day of the storm the writer was at his home in Fennville, Michigan. He received a telephone call from Mr. James E. Rickets, General Manager who gave particulars of the storm which he said was then abating. He had stopped traffic over the bridge at 5:55 P.M. By the time of the telephone call the wind velocity had dropped to 50 miles per hour and the amplitude of waves was much reduced. The District's cars had traveled over the roadway and found no trouble. Mr. Rickets asked for an opinion about resumption of traffic. The reply in effect was that traffic could be resumed as soon as drivers would have no difficulty in holding their cars in the traffic lanes they chose to occupy.

Upon arrival at San Francisco on the following day as requested by the General Manager, the Board of Directors of the District directed the writer, in co-operation with the General Manager, to supervise the repair of all damage caused by the storm.

INSPECTION AND DAMAGE

GENERAL. Inspection of the suspension structure from anchorage to anchorage made by the writer with the aid of District employees disclosed only superficial injury except at expansion joints where the lateral system of the center span connects with the towers.

At the latter points the sliding block joints were damaged so that replacement of one bearing plate and the block was necessary at the San Francisco Tower while at the Marin Tower replacement of both bearing plates and the block was necessary. These various items will be discussed in detail hereinafter.

MARIN PIER. A visual inspection of the Marin or North Pier shows no deterioration. This pier was built on a small island of rock. Excavation for the pier left a rim of this rock of varying height around the pier except at the southwest corner where the excavation was deepened thirteen and one-half ($13\frac{1}{2}$) feet in order to reach a solid bed. Small portions of this rim project above the water. Tide was out at the time of the writer's inspection and there was no apparent change in the visible part of the rock. Unquestionably the foundation is sound.

SAN FRANCISCO PIER. The San Francisco or South Pier and Fender is unaffected by the storm. There has been no appreciable deterioration here since construction.

ANCHORAGES. The Marin cable anchorage was in no way affected. It is in perfect condition. An excellent job of maintenance has been carried out on this vital part of the structure. Here there has been no deterioration since the date of its construction.

The Marin Anchorage Housing was in no way affected by the storm. However, as has been previously reported repairs were needed at floor stringer seats on the back wall of Pylon N1 and on the expansion sides of Bents designated as Nos. 3, 6 and 9. The need for these repairs was not related to the storm.

The San Francisco cable anchorage was in no way affected by the storm and is in perfect condition. Maintenance has been of the same high order as that at the Marin Anchorage.

PYLONS N1 AND S1. Pylon N1 located at the shore end of the Marin side span houses two vital elements of the structure — the cable tiedowns and the lateral connections for the side span. Also located here are the rocker links which maintain the end of the span at the required elevation. These are all in perfect condition and have not been affected by the storm.

Dirt from the roadway accumulates rapidly here and requires more frequent cleaning and painting. No deterioration is evident.

Sheet metal housings around the cables where they enter the Pylon show result of corrosion from the inside which surfaces are inaccessible for painting. These housings are purely superficial and at the time of this inspection nothing needs to be done. They are intended to be replaced when necessary.

Pylon S1, located at the shore end of the San Francisco side span serves the same purpose at the south end of the bridge as does Pylon N1 at the north end. The comments made above for Pylon N1 apply here also except that a very small chip of bronze has been broken off from a rubbing block which is in sliding contact with the tiedown cable band on the east cable. This injury was probably done during the storm. It is of no consequence and needs no repair.

CABLES. Aside from a few spots where paint was loosened or removed, the cables were not injured by the storm.

CABLE BANDS. Cable bands were not damaged. Chafing of suspender ropes in the grooves at a few points near mid-span, where suspenders are short, rubbed off some paint. The restoration of paint is a task of the regular maintenance force.

On the east cable of the center span the first regular band south of the Marin Tower showed evidence of a movement of about one-sixteenth ($1/16$) inch along the cable. This is unimportant. There was no movement of other bands with respect to the cables.

SUSPENDERS. Suspender ropes were not damaged. At a few points the suspender spacing clamps just below the cables were displaced upward. These have all been returned to their normal position.

At numerous points where the suspender ropes pass through the top chord cover plates the wire wrapping used to protect the ropes against chafing (or in some cases the steel shields added since construction for this same purpose) were disturbed and many new shields have been added by the District's maintenance crew, or old shields have been adjusted in position so that all points are protected. Much of this has been done in the past as a regular maintenance matter.

SUSPENDED STRUCTURES. The stiffening trusses, floorbeams, stringers, laterals and handrailing were undamaged.

Sidewalk concrete in a few places along the outer edges where railing posts pass through it show bits of concrete cracked off. This in no way affects the serviceability

of the sidewalks and repairs of the same are not necessary. Roadway concrete was not damaged.

Concrete bases under lamp posts located 350 feet each side of mid-span on the east side of the roadway were broken when the cable rubbed against these posts. These bases were rebuilt by the District's maintenance crew.

TOWERS. A thorough inspection of the towers with particular attention to the tops showed no damage. Saddle castings and connections for same to the tower tops were in perfect condition. Connections of transverse top struts between tower legs showed no strain whatever. Bracing between tower legs immediately below the roadway where lateral forces from the suspended spans are unloaded was in perfect condition.

Rocker links and bearings for the same at both towers were undamaged.

LATERAL CONNECTIONS OF SUSPENDED SPANS TO TOWERS. The lateral connections of side-spans to the towers in both instances were undamaged.

The lateral connections of center span to the towers, however, suffered considerable damage. The construction of this connection located just beneath the bridge floor on the longitudinal center line of the bridge is arranged so that the lateral system transfers its load through a sliding block to a large slotted casting which is fixed to the tower.

The ends of the sliding block are turned down to a twelve (12) inch diameter to fit into bronze bushings carried by the lateral plates. The latter lie in horizontal planes. This permits angular motion of the lateral plates about the vertical axis of the block and thus accommodates the angular movements of the end of the span as it deflects horizontally.

The block is in sliding contact with the vertical faces of the bearing plates which line the casting on each side of the slot. This gives freedom for longitudinal movement of the center span within the limits of the slot and also accommodates angular motion in a vertical plane. The bearing plates were of bronze and the sliding block of Allegheny Metal 33.

During the storm, the rapid movement of the block along the unlubricated bearing plates (pressure was against the plates on the east side of the slot) generated considerable heat. The edges of the block cut into the east bearing plate and actually shaved off strips of the bronze. Eventually portions of the bearing plate tore loose from the casting. The face of the casting itself along the east side of the slot was worn away in places as was also the block.

In general the repair job at each end of the center span was to remove the damaged bearing plates; restore the worn places on the casting; install new bearing plates and new blocks.

To give access to the work the upper lateral plate was removed. The block was then lifted out; the damaged bearing plate removed; the casting built up by arc welding; finished down to required plane; new bearing plate installed and matching holes for through bolts drilled through the casting to hold the bearing plates in position.

(The original bearing plates were held in place by tap screws.) The new block was then installed and the upper lateral plate was put back in place. (At the Marin Tower both bearing plates were replaced.)

The above operations started on December 9, 1951 and were completed at the San Francisco Tower on January 4, 1952. Work at the Marin Tower was completed about January 22, 1952. Much of the work was performed during a period when high winds and rainfall were far in excess of normal. On many days no work could be done.

At the beginning of work each day it was necessary to remove the upper lateral plate and the block. Each night these parts were reassembled. The two center lanes were blocked to traffic during working hours but each evening the barricades were removed so that at night all lanes were opened for traffic.

In order to effect repairs and restore normal functioning as early as possible, steel bearing plates were installed and blocks of Forged Steel SAE 1040 annealed were used. In the new detail provision has been made for lubrication of the sliding surfaces and rounding off the edges. The materials used are not so resistant to corrosion and wear as the materials formerly employed, but they will serve properly for a long time and until parts manufactured of the preferred materials can become available, at which time the change over can be made quickly, since the details employed have been designed with that in mind.

The original sliding blocks have been carefully crated and marked for future utilization.

The work was done by the Judson Pacific-Murphy Corporation, Emeryville, California, under the direction of the general manager and the writer. Whereas the purpose of this report is to cover the storm damage to the Bridge and its repair, it seemed desirable to include for the record factual information regarding the storm and the movements of the Bridge.

Steel for half of project — 2,500 tons — was trucked to base of Marin Tower where contractor maintained a five-day stockpile.





RECOMMENDATIONS

(From the report of Clifford E. Paine, January 18, 1952)

"Repair of storm damage to the Bridge has been practically completed.

"For the first time in its fourteen years of service, the character, intensity, and direction of the wind resulted in movements of the bridge floor of important magnitude. It is believed that much can be done to improve this behavior.

"The addition of a system of lateral bracing in the plane of the bottom chords in conjunction with cross bracing in vertical planes at regular intervals would be highly beneficial.

"Other possibilities include the employment of open grating floor in some sidewalk areas and the use of damping devices. The field of possibilities is large and should be thoroughly explored.

"The writer would participate in the investigation if the Board so desires, on condition that he be authorized to consult and collaborate with two or more consulting engineers experienced in this field."

It is therefore recommended that:

1. The writer be authorized:

- (a) To study the feasibility of adding a lateral bracing system in the plane of the bottom chords of the stiffening trusses.
- (b) To study the effects of such a bracing system on the behavior of the Bridge — taking into consideration any net change in weight of the suspended structure and any resulting change in the magnitude and distribution of impressed wind forces to all parts affected.
- (c) To study the feasibility and desirability of employing an open grating type of floor on some sidewalk areas.
- (d) To study the feasibility and desirability of adding damping devices to restrain and cushion longitudinal movements of the center span.
- (e) To study the feasibility and desirability of making any modifications of, or addition to the structure which, as the study develops, might appear to be necessary or advisable.
- (f) To engage (on behalf of the Golden Gate Bridge and Highway District) two or more consulting engineers of recognized standing and having direct experience in the problems involved to collaborate with the writer in a review of these studies and the preparation of a report setting forth the conclusions and recommendations jointly agreed upon.

2. The District arrange with the manufacturers of vibration recording instruments to have those on the center span modified so that the driving mechanism of the recording drum can be instantly changed to increase the speed of the chart to about three times the normal speed. This will give graphs which can be interpreted with greater certainty.

In use, the instruments would operate normally as at present but whenever the vibrations became of important magnitude the shift would be made (either automatically or semi-automatically). In this way, the charts would cover the same period of time as at present except when it becomes advantageous to increase the speed of the drum.

3. One of the vibration recording instruments be moved from the North side-span to the North quarter point of the center span — east side.

ENGINEERING STUDY INITIATED. In his report quoted above, the writer advised that the addition of a system of bottom lateral bracing, in conjunction with cross bracing in vertical planes at regular intervals, would be highly beneficial. He was authorized by the Board of Directors of the Golden Gate Bridge and Highway District to proceed with the studies recommended in his report. He developed the plans therefor in preparation for their review by two consulting engineers he had been authorized to name for the purpose of collaborating with him. On April 25, 1952, as recommended by the writer, the Board of Directors confirmed the appointment of Mr. Othmar H. Ammann and Charles E. Andrew, Consulting Engineers, to form with the writer a Board of Engineers. The Board of Engineers thus constituted held its first meeting in Seattle, Washington, on May 15, 1952.

CHARACTER OF PROBLEM

In considering ways and means to improve the existing structure so as to eliminate all objectionable motion under severest wind action, the Board of Engineers realized that they were confronted by a problem which could not be solved with mathematical certainty.

The problem is an extremely complex one. Its solution is dependent upon many varied factors, some of which, such as the nature of wind action on the actual structure and damping resistance of the structure, are uncertain and have not as yet been adequately explored.

It is largely due to these uncertain and unexplored factors that, despite volumes of theories developed since the failure of the Tacoma Narrows Bridge and of extensive research work and laboratory experiments carried out in this country and elsewhere, we are still dependent to a large extent upon judgment based on the comparative behavior of existing similar structures, supplemented by trends indicated by model experiments.

Moreover, while in the case of a new suspension bridge various means can be applied so as to give practical assurance of ample resistance against dynamic wind action, there are limitations to the use of similarly effective means in an existing large structure, such as the Golden Gate Bridge, without involving changes of major extent and at exorbitant cost.



It was considered desirable to carry out a program of so-called section model tests, that is tests made on a scale model of a short section of the suspended floor structure, to supplement knowledge which had been derived from similar tests made previously. Only limited information can be obtained from such section model tests. Doubtless a scale model of the entire bridge would have permitted a somewhat closer approach to a solution. Even a full model, however, has its limitations. It would involve a relatively large expenditure and its construction, together with an adequate program of tests on the same, would consume more than a year's time.

Despite the complexity of the problem and the limited time and means available it is the judgment of the Engineering Board that their studies, supplemented by present knowledge of behavior of existing bridges as well as from extensive research work carried out here and elsewhere, make an adequate solution of the problem possible.

IMPROVEMENTS CONSIDERED

The following means to improve the present (1953) condition of the Golden Gate Bridge have been given consideration in this study. All have been applied in the construction of new large bridges or in the improvement of existing ones and certain data are available relative to their effect.

ADDITION OF BOTTOM LATERAL SYSTEM. In his study of the Failure of the Tacoma Narrows Bridge the writer discovered that in any suspended structure having trusses on two sides, lateral bracing on either top or bottom side and with fourth side open the closing of the fourth side by installation of a lateral bracing system was by far the most effective measure that could be taken to increase torsional resistance. In his report to the Board of Directors of the Golden Gate Bridge and Highway District dated January 18, 1952 he therefore stated that "The addition of a system of lateral bracing in the plane of the bottom chords in conjunction with cross bracing in vertical planes at regular intervals would be highly beneficial."

The Golden Gate Bridge, like many other suspension bridges, was designed and built with a single system of lateral bracing immediately below the roadway in the plane of the top chords of the stiffening trusses. Use of a single lateral system left the bottom side of the suspended structure open and hence its torsional resistance was low.

It is evident from observations on the Golden Gate Bridge, in particular those taken from the instruments on the bridge during the storm of December 1, 1951, that the severest motions were distinctly of a torsional character combined with vertical motions.

This tendency towards torsional oscillations is also evidenced by tests which had been made on models of the Golden Gate Bridge and several other truss-stiffened suspension bridges, in particular models of the rebuilt Tacoma Narrows Bridge and



of the proposed bridge across the Severn in England. Both the latter bridges have been designed with a double system of lateral bracing to resist this tendency.

Double lateral systems have also been applied in the design of the Delaware Memorial Bridge at Wilmington.

Neither the new Tacoma Narrows Bridge, completed in 1950, nor the Delaware Memorial Bridge, completed in 1951, have so far experienced noticeable motions under severe wind action.

Tests made at Princeton University in connection with the construction of the Delaware Memorial Bridge closely checked theoretical calculations to the effect that the addition of a lower lateral system increased the torsional rigidity of the suspended floor structure of that bridge about 20 times.

Similar calculations made for the Golden Gate Bridge, as reported hereinafter, show that the torsional rigidity of the floor structure by the addition of the proposed lower lateral system increased about 35 times, and that the torsional resistance of the entire suspended structure, cables and floor structure, is 2.75 times the corresponding resistance without lower laterals.

As verified by the section model tests hereinafter reported, this stiffening effects a substantial increase of the critical wind velocity at which torsional motions of the structure will be initiated. This increased resistance to torsional motions is accompanied by increased structural damping produced by the addition of the lower lateral system.

These considerations lead to the reasonable conclusion that the addition of the bottom lateral system will so materially improve the resistance of the Golden Gate Bridge to wind action that no other improvements may become necessary.

CENTER STAYS. Effective center stays have the purpose of preventing the racking, opposite longitudinal motions of cables and floor structure which result from asymmetric distortions of the oscillating structure under dynamic wind action.

Such devices were introduced in the original Tacoma Narrows Bridge after its completion in an attempt to steady the structure. There can be little doubt that as long as the center stays remained intact they prevented the development of the severe torsional motions which led to the ultimate destruction of the suspended structure. Those center stays and in particular their connections to the cables were, however, inadequate to resist the forces which developed. Moreover their influence in reducing vertical asymmetric motions proved to be limited.

More effective center stays were introduced in the Bronx-Whitestone Bridge which experienced oscillations of moderate amplitudes after its completion in 1939. While they are unquestionably effective in preventing the development of torsional motions, and have had a noticeably favorable influence on vertical oscillations, the latter influence alone did not prove adequate to steady the structure satisfactorily against vertical motions.

A very substantial system of center stays has also been made part of the design of the new Tacoma Bridge. Their favorable influence was indicated by tests made on the



full model of that bridge, but it would be impossible to assign a quantitative value to their effect in comparison to the effect of other means employed in that design to secure steadiness, vertically and torsionally. It is believed that, while the influence of center stays in preventing asymmetric torsional motions is unquestionably very appreciable, in fact positive so long as they remain intact, they are an additional, but not necessarily essential safeguard.

The practicability of introducing center stays in the Golden Gate Bridge was studied and it was concluded that they should be installed only as a further step in case the addition of a bottom lateral system should prove not entirely adequate to prevent objectionable motions.

On the Bronx-Whitestone Bridge and the new Tacoma Narrows Bridge, center stays were used in combination with damping devices at each end of the main span. No practical way was found of applying such end damping devices to the Golden Gate Bridge and it is not now considered an essential improvement.

ALTERATIONS OF FLOOR STRUCTURE. Extensive experiments made with models in wind tunnels, notably those made in connection with the redesign of the Tacoma Narrows Bridge have revealed beyond doubt that the form of the suspended structure may have a marked effect on its behavior in wind and may contribute towards its steadiness.

The Tacoma Narrows Bridge tests revealed in particular that certain openings in the deck structure can be very effective in this respect and as a result such openings were provided in the design of that bridge in the form of strips of open grating flooring between solid concrete slabs. These and other model tests indicated that in general openings in the deck structure tend to be beneficial.

Based on these findings it was believed that in the case of the Golden Gate Bridge beneficial effects would be produced by replacing the 11 ft. wide solid deck footwalks partially or entirely by open steel gratings. The exploration of these effects was the principal reason for carrying out a program of section model tests by which a measure of influence of such openings and other structural changes in the floor structure could be obtained.

The results of these tests described hereinafter, while indicating the possibility of improvement, were disappointing in that they did not reveal under all reasonably possible wind conditions a favorable effect of sufficient magnitude to justify the extensive and costly structural alterations of the deck structure before ascertaining whether other improvements such as the addition of a bottom lateral system and possibly center stays will be adequate.

Judging from the results of the Tacoma Narrows Bridge tests it is reasonable to assume that more extensive openings, including parts of the roadway deck, would be more effective, but such changes would be difficult to accomplish and very costly. For these reasons the program of section model tests did not include such changes.

If the addition of bottom laterals and possibly the introduction of center stays

should be inadequate to prevent objectionable motions, then the expedient of opening the sidewalks and possibly other changes in form should be given further study.

OTHER STAY SYSTEMS. In several suspension bridges which experienced excessive motions in wind, various systems of diagonal wire-rope stays were introduced, connecting the tower tops or points along the cables with points along the floor structure. A sufficient number of such diagonal stays properly placed may be effective in steadying the structure.

Such stay systems, however render the entire suspension system very complex and highly indeterminate in stress action. The design of a long-span suspension bridge, which for economic reasons has to be relatively flexible, anticipates complete freedom of action for the cables except for intended stiffening that results from its connection to the stiffening trusses by vertical suspender ropes.

It is desirable to attain this freedom of action as far as possible. The provision of diagonal stays tying the cables to the stiffening trusses restrains the cables so that they no longer function in the free and simple manner anticipated in the design. For this reason the need for such stays must be predominating to justify their use.

The degree of restraint exercised by such stays in resisting dynamic wind action can only be reliably determined either by trial on the actual structure or by elaborate tests on a full model of the bridge.

To be effective in the Golden Gate Bridge such a system might be very expensive to install and would adversely affect its good appearance.

SECTION MODEL TESTS

As previously stated herein the problems of determining what motions will occur in any suspension bridge due to dynamic wind action is a complex one. For any given structure certain influencing factors, such as period of vibration, characteristics of wave forms, etc., are subject to mathematical solution because they depend only on the known elastic properties of the structure. Other factors, such as the efficiency of the wind action on the actual bridge and the ability of the structure to dissipate energy, can be evaluated only on assumptions which must be derived from field observations on and experience with existing structures supplemented by judgment.

Fortunately the recorded observations on the Golden Gate Bridge furnish much dependable information in this regard. This knowledge derived from actual structures must be supplemented by judicious interpretation of experiments with models in wind tunnels.

TYPES OF MODELS. Two types of models are available, viz; full models and section models.

A full model is one which represents the full structure to some scale and simulates it elastically. Such a model is costly, must be tested in a specially adapted wind tunnel, and its construction consumes much time. It has an advantage over the section

model in that wind can be applied over any portion or all of its length. Its periods of vibration and all other characteristics of the motions produced by wind can be visually observed and measured. Parallel tests made on full models and corresponding section models have demonstrated, however, that certain useful conclusions can be derived from section models.

A section model is one which represents only a portion of the suspended structure and simulates the bridge to some scale geometrically. Elastic similitude is obtained by mounting the section model with the aid of springs designed to simulate the periods of vibration of the actual structure. The latter must be computed. Section models are mounted in the wind stream in two manners, one is termed "torsional mount" and the other "free spring mount." They are described below.

On account of lack of time and the great cost involved, tests for the Golden Gate Bridge were confined to section models. A scale of 1 to 75 was adopted.

The program comprised tests on the structure as built, tests on the same structure with a bottom lateral system added and tests simulating various modifications of the sidewalk structure combined with the addition of bottom laterals.

Following are definitions of some conventional terms used in model tests:

A section model with "free spring mount" is one which is suspended with the aid of springs designed to impart to the model the computed periods of vibration. The springs are so attached as to allow three degrees of freedom of motion, i.e., vertical, torsional and horizontal.

A "torsional mount" section model is one in which the springs are designed to impart to the model the computed period of torsional vibration. In this case the model is pinned at its ends in such a manner as to allow only one degree of freedom of motion, i.e., torsional about a longitudinal axis.

"Angle of attack" designates the inclination of the wind current in the horizontal plane. Positive angles designate upward and negative angles downward wind currents.

"Decrement" as used in these tests is a function which measures the ability of the structure to dissipate energy and damp out motion. Decrement includes both structural and air damping.

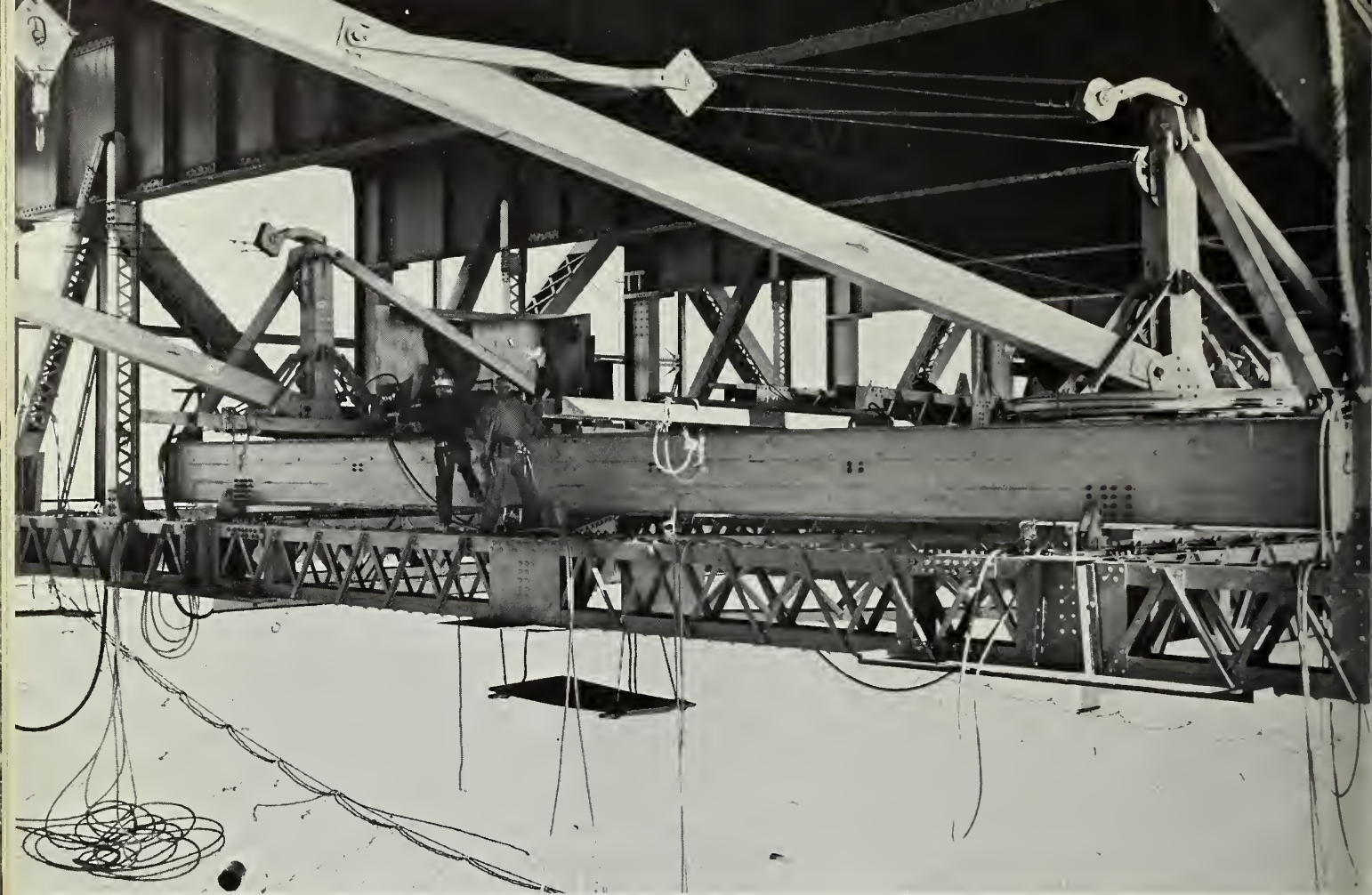
"Critical velocity" is the velocity of the natural wind at which any particular type of vibration is initiated.

APPLICATION OF MODEL TESTS TO THE ORIGINAL AND MODIFIED STRUCTURES.

Any predictions of the behavior of the actual structure of the Golden Gate Bridge, as built or as modified, from the model tests alone can only be qualitative and not quantitative on account of the previously mentioned unknown factors. Among these are the decrements of the structure as originally built and in the various proposed modified forms, also the inefficiency of the wind in nature as compared with the almost perfect wind used in model testing.

In the laboratory a wind stream is created which is constant in velocity and in direction and uniform over the full length of the section model. In nature, however,





Chicago booms with a six-ton, 45-foot radius capability were mounted to a carrier operated on the new, permanent rail system once steady erection was under way. Carrier capacity was 20 tons at 50 feet per minute up to the maximum 3% grade of structure. Two erection crews started, one at each tower, and worked toward midspan. Side spans were erected last. Initial two panels at towers provided working platform for remainder of operation and were lifted in place by truck cranes, on bridge roadway, with booms over the side.

the wind is constantly changing in velocity over short periods of time. It strikes the structure at different angles and with different velocities at different locations throughout the great length of the bridge. In full model tests of the Tacoma Narrows Bridge, in which the angle of attack could be changed on various portions of the model in any one test, it was clearly demonstrated that such changes would immediately damp out or reduce violent motions. A change in velocity without change in direction has the same effect because for each velocity the action upon the structure is different. A gusty wind therefore has a damping tendency and becomes less efficient in producing motion than a wind of constant velocity as used in model tests.

For these reasons it must be concluded that the critical wind velocity on the actual structure is substantially higher than that indicated by the results of model tests. Just how much this difference is must be largely a matter of judgment.

In the case of the Golden Gate Bridge there are certain actual occurrences which serve as a guide to such judgment as follows: During the storm of December 1, 1951, which is described under "History," the center span was oscillating in a somewhat mixed first symmetric torsional motion with a double amplitude of 22 inches at 5:30

P.M. with wind velocity of 50 mph. and a period of vibration of 8 cycles per minute. Within a few minutes the wind velocity started to increase and the motion of the center span changed to a mixed first asymmetric motion (a coupled vertical and torsional motion in which the main span was twisting in two segments). Amplitudes in this motion increased rapidly as the velocity increased to 69 mph. at 5:55 P.M. with a period of 16.8 cycles per minute and a maximum double amplitude of 130 inches. At this time the wind velocity started to decrease and with it the amplitude until, a few minutes after the wind had reduced to 50 mph., the double amplitude was about 20 inches.

It is reasonable to conclude that for the wind condition existing at approximately 5:30 P.M. and again at approximately 6:10 P.M. the actual critical velocity for the first asymmetric motion of the structure as built is approximately 50 miles per hour.

Referring again to "History" in this Supplement, a storm on June 6, 1950 developed a 56 mile wind from the west causing a maximum amplitude of 45 inches at mid span in the first symmetric torsional mode of motion. For brief periods at this velocity the motion showed a tendency to change to the first asymmetric torsional form. Apparently the critical velocity in this storm for the first symmetric torsion was 56 mph instead of 50 mph, as observed during the December 1, 1951 storm, and the efficiency of the wind was accordingly less.

From this bridge history it seems logical to assume that the critical velocity in the first asymmetric motion is not less than 50 mph.

PREDICTION OF CRITICAL VELOCITY FOR THE MODIFIED STRUCTURE WITH BOTTOM LATERALS. As stated herein before, model tests can give only qualitative results unless the actual decrement of the structure in any given form and the efficiency factor of the wind are known.

The problem of predicting the critical velocity for the present bridge with bottom laterals depends upon the difference between the decrements rather than the actual magnitude of the decrements. It would therefore be simplified if we knew the difference between the respective decrements of the bridge as built and that of the bridge with bottom laterals added.

If we assume the decrement of the bridge as built to be the same as that of the modified structure, then the degree of improvement of the modified structure over the original one would be measured by the difference in critical velocities of the respective response curves. We can reliably assume, however, that the decrements of the original and the modified structure are not the same because of the appreciably greater torsional stiffness produced by the addition of bottom laterals.

To reduce model tests to quantitative results we must assume decrements which in our judgment actually existed in the original structure and will exist in the modified structure, as a first step toward predicting the critical velocity of the bridge with bottom laterals added. A decrement of 0.0375 has been assumed for the original structure and 0.057 for the modified structure, a difference of 0.0195.

This judgment is based on the fact that the torsional stiffness of the suspended floor structure will be increased by the bottom laterals approximately 34 times, the torsional stiffness of the entire suspended structure, including cables, will be increased approximately 2.7 times and the frequency of vibration will increase from 7.26 to 12.6. Making the above reasonable assumptions constitutes the first step in approximating the critical velocity for the modified structure.

The critical velocity of the section model of the original structure as derived from the section model test and with a decrement of 0.0375 was found to be 24.9 mph. The critical velocity of the section model of the same structure with bottom laterals added as derived from the section model tests and with a decrement of 0.057 is 51.4 mph. The difference between these two critical velocities is 26.5 mph. This difference gives a measure of the improvement gained by adding a bottom lateral system.

The inefficiency of the natural wind is a second factor to be applied in predicting the critical velocity for any structure under actual wind from tests on a section model of that structure in ideal wind.

Appraisal of this factor is also a matter of judgment based on the past performance of the original bridge. The storms of June 6, 1950, and December 1, 1951, clearly reveal the actual critical velocity in the first asymmetric motion developed in those particular storms. These critical velocities were 56 and 50 mph, respectively, an average of 53 mph.

There is no assurance that future storms will confirm these critical velocities. Different angles of attack and different wind patterns and velocities may occur along the bridge axis which will modify the action. It may reasonably be expected, however, that future severe storms will be similar in character to those experienced to date and that the critical velocities will not vary greatly from those experienced heretofore.

It has been conservatively assumed that 48 mph is the minimum actual critical velocity for the original bridge (first asymmetric motion). The corresponding critical velocity derived from the section model is found to be 24.9 mph, or 23.1 mph less than the above critical velocity derived from those observed on the original bridge.

By combining the two factors, the one due to added stiffness and the one due to wind inefficiency, and applying them to the results from model tests, it is possible to predict quantitatively the approximate critical velocity for the actual bridge with bottom laterals added. Three different methods may be used.

1. Using the critical velocity of 24.9 mph of bridge as built, which was derived from the model test with decrement of 0.0375, adding thereto the above assumed minimum increase of 23.1 mph for wind inefficiency and the above increase of 26.5 mph for greater stiffness, gives a critical velocity for the actual bridge with bottom laterals of 74.5 mph.
2. Using the critical velocity derived from the model test with decrement of 0.057, i.e. 51.4 mph, and increasing this for wind inefficiency, i.e. 24.9 mph, gives 76.3 mph as the critical velocity of the prototype with bottom laterals.

3. Using the average critical velocity of the bridge as built which was derived from observations in the storms of December 1, 1951 and June 6, 1950, which presumably reflects wind inefficiency, and adding the effects of greater stiffness gives $53.0 + 26.5 = 79.5$ as the critical velocity of the actual structure with bottom laterals.

It is therefore reasonable to expect that the critical wind velocity for the first asymmetric mode of motion of the present bridge with bottom laterals will be between about 75 and 80 miles per hour. Instantaneous gusts varying from 68 to 85 miles per hour may occur during storms of such velocities without aggravating the behavior of the structure.

It is not expected that adding a bottom lateral system will eliminate all movements of the bridge in the first asymmetric mode of motion. Since the bridge was finished this type of motion has occurred at relatively low wind velocities. Its maximum double amplitude of 45 inches was reached during the storm of June 6, 1950 when the wind reached a top velocity of 56 mph. At this velocity the first asymmetric motion appeared during brief periods. First asymmetric motion of this magnitude is not injurious to the structure but is somewhat objectionable in its effect on traffic.

The addition of a bottom lateral system tends to decrease the amplitudes of this motion approximately in the ratio of the periods of vibration of the bridge as built and of the same with bottom laterals added. This ratio is 9.61 to 11.08 and the decrease approximately 15%.

The above findings justify the recommendation to add a bottom lateral system as the first step in protecting the bridge against harmful or objectionable motions.

EFFECT OF CENTER STAYS. In case the addition of bottom laterals should prove insufficient, it has been recommended, as a second step, the incorporation of a system of center stays as described elsewhere hereinbefore. The effect of such stays could not be tested on section models.

Tests on the full model of the Tacoma Narrows Bridge demonstrated that as long as center ties remain intact no asymmetric motion occurred and it is safe to assume that the same would apply to the Golden Gate Bridge with effective center stays.

EFFECT OF OPENING FLOOR STRUCTURE. Model tests made in connection with the redesign of the Tacoma Narrows Bridge indicated clearly that opening up parts of the solid deck structure can materially improve the behavior of the structure and consequently such openings in the form of longitudinal strips of open grating flooring between the lanes of traffic were used in the new bridge.

Having in mind the practicability of providing such openings in the Golden Gate Bridge by replacing all or part of the solid slab sidewalk with open grating and of making other possible structural changes near the edges of the floor structure which would improve the behavior, a number of section model tests were made to determine the effect of such changes.

The tests indicated indeed that certain of such changes would have a beneficial effect.



One of the structural conditions tested, namely with the entire sidewalk slabs replaced by gratings and the sidewalk stringers and the curbs removed, it was indicated that the critical velocity of the prototype with bottom laterals would be raised to 60.7 mph.

Adding to this the previously derived allowance of 23.1 mph for wind inefficiency one arrives at a possible critical velocity of the actual structure with bottom laterals and with the aforementioned changes in floor structure of $60.7 + 23.1 = 83.8$ mph. Wind gusts might well be above 90 mph.

Such an arrangement would probably also reduce the amplitudes of motions in the first symmetric mode.

BOTTOM LATERAL SYSTEM

The study relating to the addition of a bottom lateral system had as its objectives to determine its feasibility structurally and its desirability as a means to improve the behavior of the bridge.

The question of feasibility hinges primarily on the practicability, from a mechanical standpoint, of installing an adequate system of bracing.

The question of desirability hinges on the effect which the bracing will have on the behavior of the structure, on its dead weight and on its cost.

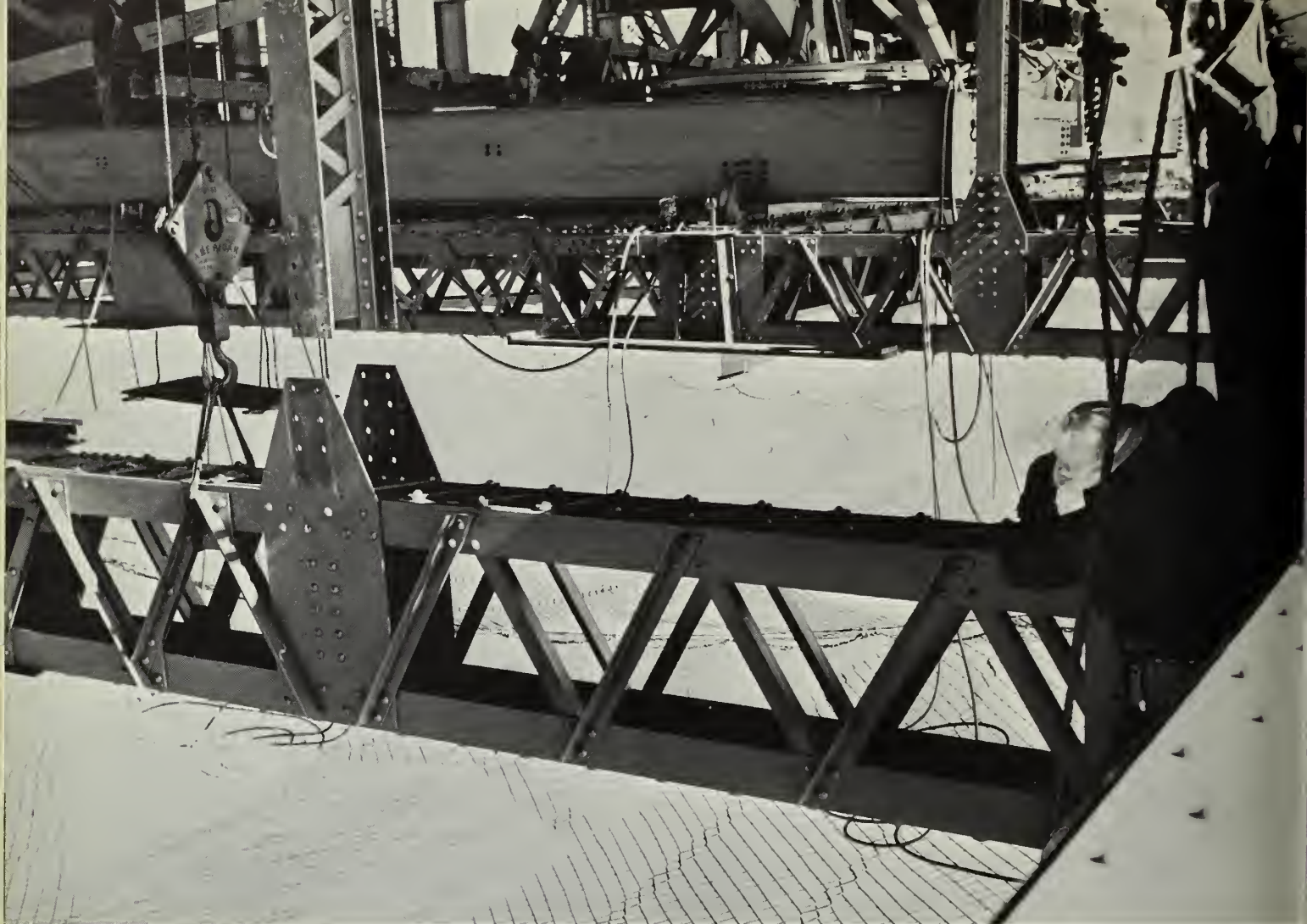
DESCRIPTION. The general arrangement of the proposed bracing is shown on Plates SI and SII. It will be noted that a panel of lateral bracing is about 100 ft. long, thus extending over four 25-ft. panels of stiffening trusses. Diagonal bracing members are connected to bottom chords of stiffening trusses at panel points chosen to miss main gusset connections for stiffening truss diagonals.

The lateral connection plates were placed directly against the top and bottom sides of the chords. A large part of the connection holes in the chords were provided by removing 1-in. dia. rivets. A few holes had to be drilled at each connection. Field connections were made by 1-in. dia. high tensile bolts. Transverse struts made up as latticed girders are placed at every truss panel point. These form the bottom members of the cross-frames and also support the longitudinal track girders. The latter carry rails upon which the traveling maintenance scaffolds operate.

At each end of each span the cross-frames are designed to carry bottom lateral stresses up to the plane of the top laterals and thence to the towers or pylons.

The installation of a bottom lateral system as described above proved to be entirely adequate and feasible.

STRESSES AND SECTIONS. The addition of bottom laterals converted the suspended structure into one which is much more resistant to torsion. This increased torsional stiffness results in a somewhat different stress pattern and these stresses have to be provided for wherever they occur. The lightest lateral members of the top lateral system are box sections composed of four angles $4 \times 4 \times 7/16$ laced on four sides. This



End sections of 90-foot transverse struts were second in erection sequence.

minimum section occurs over the middle portion of the center span. Here stresses from torsional deformation typical of the first asymmetric mode of vibration are greatest. Obviously nothing could be gained by employing a heavier section for the bottom laterals unless the top laterals were also increased.

Calculations show that this minimum lateral section is sufficient to withstand stresses induced by a torsional force of 18,000 ft.lb. per foot of center span applied in one direction over the south half and in the opposite direction over the north half. In terms of asymmetric live load this is the torsion that would be obtained by loading only the southwest and northeast quarters of the roadway of the center span with 40 lb. per sq. ft. of roadway area. This torsional resistance is sufficient for all purposes and a bottom lateral system thus composed is adequate.

EFFECT ON STRUCTURE. Addition of the bottom lateral bracing affects the bridge in many ways. It is necessary to explore and evaluate several of these effects in order to be certain that the benefits are not obtained through a sacrifice of some essential quality of the structure. The effects fall into two categories: (1) that which the added torsional resistance has on the behavior of the structure, and (2) that of the added weight on the major load carrying elements of the bridge.

In one comparison made, the suspended structure of the center span was considered cut loose from the cables and subjected to a torsional force uniformly distributed over the south half of the span and a like torsional force acting in the opposite direction over the north half. The structure without bottom laterals showed an angular rotation at the quarterpoint 34.8 times greater than that of the structure with bottom laterals.

A similar comparison was made with the structure suspended from the cables and this showed for the assembled structure without bottom laterals an angular rotation at the quarterpoint 2.7 times greater than that for the structure with bottom laterals.

By adding the bottom laterals the calculated frequency of the center span in the first asymmetric mode of torsional vibration is raised from 7.26 to 12.6 cycles per minute. In the first symmetric mode of torsional vibration the frequency is raised from 9.61 to 11.08 cycles per minute (neglecting tower stiffness). These increases in frequency are noteworthy, especially for the first asymmetric torsional mode.

Torsional forces applied to the structure without bottom laterals are carried almost entirely by the cables. In the structure with bottom laterals such torsional forces are so divided between the cables and the suspended structure that the former carries 36% while the latter carries 64%. This is another reflection of the reduced angular rotation referred to above.

Since the structure with bottom laterals resists torsional forces it must be designed accordingly. The top and bottom lateral systems, including their respective stiffening truss chords, constitute trusses lying in horizontal planes and they are subject to shearing forces that arise from any torsional force applied to the suspended structure. This may come from live load or wind. Calculations were made for an asymmetric live load of 40 lb. per sq. ft. on diagonally opposite quarters of the main span roadway. This produces a torsional force, mentioned before, of 18,000 ft. lb. per lineal foot of bridge. Sixty-four percent of this torsion will be carried by the suspended structure. The shearing force thus produced in the top and bottom horizontal trusses results in maximum stresses of $\pm 176,000$ lb. in the diagonals of the end panels and at mid-span. The laterals have ample section to resist this stress safely. Stresses in chords are negligible.

Erection of end section of transverse strut—Steel fabricator used special shop jigs to ream holes for high tensile bolts and eliminate preassembly of elements in plant. Existing bridge steel was prepared by chipping and wire buffing at connections.





Center sections of transverse struts were third in erection sequence and secured by undersized erection bolts. Center sections had milled ends.

The increased torsional resistance will have a pronounced effect on torsional oscillations of the center span. The benefits of the higher frequencies will be felt in two ways. First, it will require much higher wind velocities and more time to excite torsional oscillations especially in the first asymmetric mode. This is the mode with which one is most concerned. Second, the amplitude of oscillations will be favorably affected. Knowledge in this field has not advanced far enough to make possible a precise evaluation of these gains. Unquestionably, structural damping has been increased considerably and consequently the structure has an increased capability of dissipating energy by its oscillations. As a result, under any given wind condition in which oscillations are building up, a steady state will be reached at smaller amplitude than was the case before the bottom laterals were added.

Although a precise prediction could not be made as to the behavior of the bridge with bottom laterals in a storm, it is reasonable to believe that the structure as it now stands with bottom laterals in place will never experience objectionable oscillations and that the possibility of torsional oscillations in the first asymmetric mode are practically eliminated. This is the mode which developed maximum amplitudes in the storm of December 1, 1951.

EFFECT OF ADDED LOADS. The dead load used in the original design was 21,000 lb. per foot of bridge. The addition of bottom laterals increased the dead load by 1,400 lb. per foot and justifies an increase in longitudinal wind load to the extent of 269,000 lb. per tower shaft. The effect of the increase in weight extends to many parts of the structure.

The cable stress increase of about 5½% is amply provided for in the conservative design stress used. Furthermore, the testing laboratory reports on cable wire show that the average tensile strength of the wire was 4.7% above the average specified. There remains an ample safety margin in the cables.

The anchorages and their fastenings to the cables are sufficient to take care of the slight increase in the cable tension. The eye-bar chains which transmit the cable-pull into the concrete anchor blocks are made up of heat-treated eye-bars which, according to test reports on full size eye-bars have an average yield point 7% above the average specified. All details employed will develop the strength of the parts connected.

Reports of suspended rope tests at manufacturer's plant showed a strength at least 19% above that specified. They are more than sufficient to sustain the added weight.

Test reports show that the tie-down ropes have an ultimate strength 10% above that specified. This is more than enough to take care of the increased weight of 2.8%.

Under the influence of the added weight the tower tops will have moved 3 in. channelward. This adds to the unit stresses on the channel face of the tower.

The dead load cable reaction at the top of each tower shaft is increased by about 3,078,000 lb. Direct load unit stresses are increased accordingly.

The longitudinal wind load has been increased because of wind area added by the bottom laterals. A liberal allowance for this additional wind force is 269,000 lb. per tower shaft, applied at the floor level.

It is not necessary to add anything for the slight change in transverse wind area since the original design assumptions are ample to cover this.

The combination of unit stresses due to the above added forces accounts for increases along the extreme channel face of the tower. These increases are on the order of 1,200 to 1,900 lb. per sq. in. However, resulting total unit stresses exceed the design unit stresses only in the vicinity of two cross-struts. At one of the points the unit stress in silicon steel is 1,200 lb. per sq. in. above the design unit stress of 23,000 lb. per sq. in. At the other point the unit stress in carbon steel is 1,300 lb. per sq. in. above the design unit stress of 18,000 lb. per sq. in.

These increases in unit stresses are less than 7½% of the conservative design unit stresses. Since these result from a simultaneous occurrence of an improbable live load condition, combined with full transverse and longitudinal wind and high temperature, which combination will probably never occur, the tower design is ample to sustain the added loads.

The stiffening trusses of center and side spans are affected in a moderate degree by the added weight. Maximum unit stress increases in the chord members are about

5%. Design assumptions are sufficiently liberal to take care of this. The average yield point of the material used in the truss chords exceeds the average specified by 12%. The truss diagonals are not appreciably affected by the added weight.

COST. The installation of the bottom lateral system required about 4,750 tons of structural steel. The cost of the work, including engineering and all incidental expenses was \$3,500,000.00.

CENTER STAYS

As previously mentioned the installation of a system of center stays has been studied. Its installation is recommended only as a second, future step in case the bottom lateral system alone should prove not to be entirely adequate.

No quantitative section model tests could be made to determine the effectiveness of such stays, but from observation in actual bridges and from the tests made on the full model of the New Tacoma Bridge, it is practically certain that if made strong

Fast track mounted carrier used for main span portion of erection equaled setting speed of truck crane working from high ground on Marin side span. First five panels of Marin side span at Marin tower were erected from bridge pavement by truck cranes boomed over the side. Remainder were set from crane below.

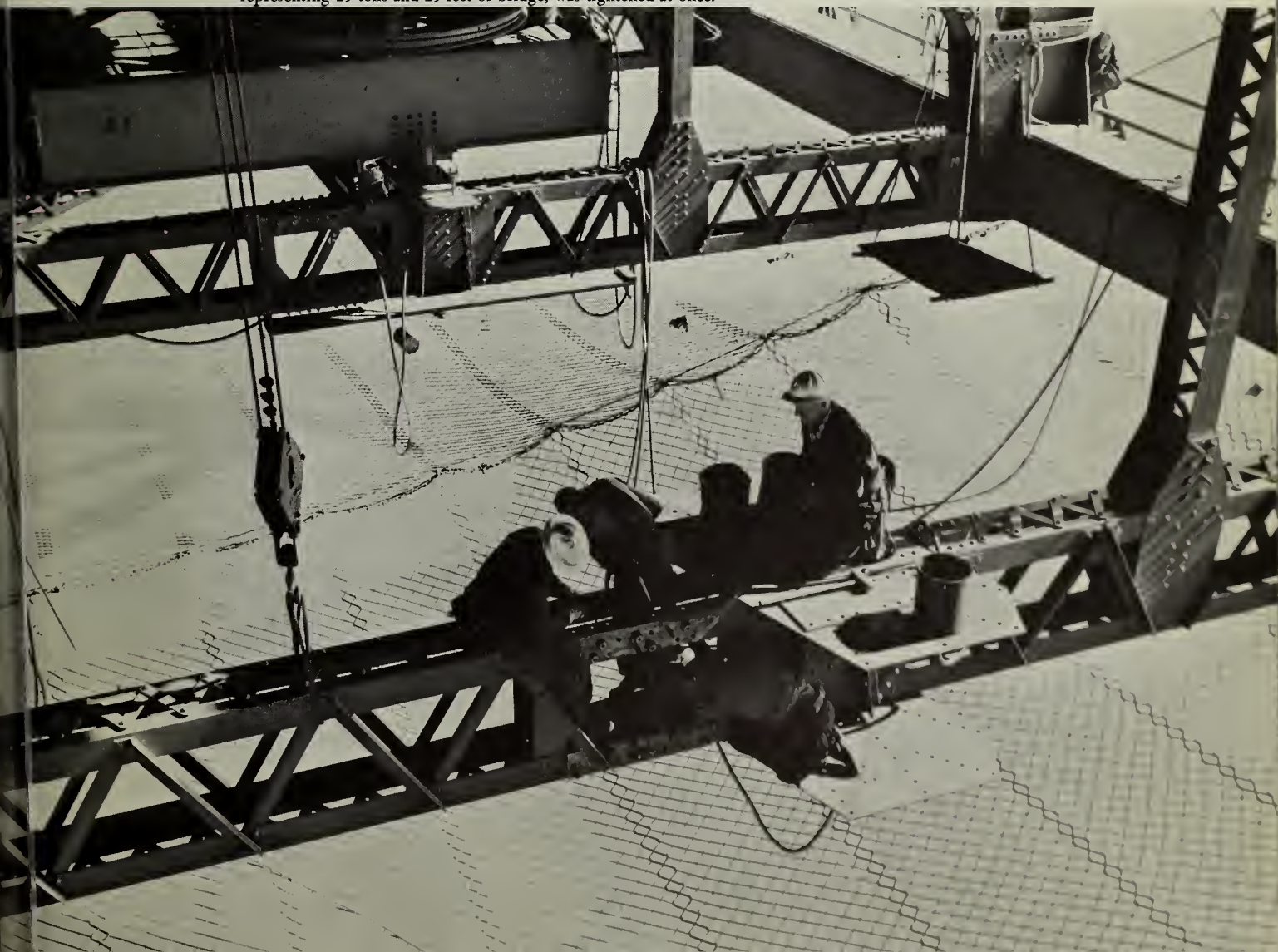


enough they will effectively prevent the development of the most critical asymmetric torsional motions of the center span, and in this respect supplement the bottom lateral system. As indicated by experience in the Whitestone Bridge they will also dampen asymmetric vertical motions and prevent the longitudinal motions of the floor in relation to the cables which were observed to be of considerable magnitude on the Golden Gate Bridge in the storm of December 1, 1951 as well as on the Whitestone Bridge prior to the installation of center stays.

The proportioning of the center stays is largely a matter of judgment. Those installed in the first Tacoma Narrows Bridge proved inadequate under the severe torsional movements.

The first temporary installation of center stays on the Whitestone Bridge, while it did not fail, indicated excessive wear and called for replacement and stronger connections. Those now installed since 1950 appear to be amply proportioned, although owing to other preventive measures they have not and probably never will be subjected to motions which might induce in them stresses even well within their safe capacity.

Erection of center section of transverse strut — Once all members were positioned, entire panel, representing 25 tons and 25 feet of bridge, was tightened at once.





Diagonal struts, lightest members in the system, were fourth in erection sequence. A complete set of diagonals occurred each 100 feet.

The Board of Engineers in their report of January 1953 state "We have satisfied ourselves that the installation of center stays, while altering somewhat the stress condition of the bridge under certain conditions of live load, would have no adverse effect on the structure."

The unqualified, unanimous recommendation of the Board of Engineers that the bottom lateral system be installed was carried out without delay and subsequently, prior to the date of this publication, there has been no objectionable movement of the suspended structure of the bridge. The installation was completed late in the year 1955.

TRAVELING SCAFFOLDS

GENERAL. For the safety of the maintenance crew and also to provide for them a place to work with greatest efficiency it was desirable to install four sets of Traveling Scaffolds — one in each side span and two in the center span. Each set comprises one Interior Scaffold, two Exterior Side Scaffolds, and one Exterior Underneath Scaffold. The two Exterior Side Scaffolds are positioned just outside the stiffening trusses on each side of the bridge.

All of the Scaffolds operate over Crane Rails and are driven by electrical power derived from generators located on the Interior Scaffold and on the Exterior Underneath Scaffold.

TRACKS. Crane Rails placed on the track girders which were incorporated in the Bottom Lateral construction form the track for the Interior Scaffolds.

Crane Rails set on rail chairs which are mounted on the top chords of the stiffening trusses provide single rail tracks for the Exterior Side Scaffolds. These rail chairs also carry a line of Trolley Beams which, engaged by rollers attached to the Scaffold, serve as a guide to keep the Scaffold in proper operating position on the rails.

INTERIOR SCAFFOLD. This Scaffold runs over the track described above. It is motor driven by electrical power derived from its own engine generator set. It moves in the clear space bounded by the bridge floor beam above, the bottom laterals below and on either side (transverse to the bridge) by vertical members which occur at each panel point (twenty-five feet spacing in the longitudinal direction of the bridge). The clear distance between these vertical members is about sixty feet.

The Interior Scaffold comprises three principal structural units, namely: one Carrier Unit and two Retractable Wings.

The Carrier as its name implies is the main unit. It has a top working deck and below deck it carries the Retractable Wings. It runs directly on the rails. Extending in a direction transverse to the bridge it has a tubular space of rectangular cross section. At the bottom of this space rails are provided on which the Retractable Wings can be rolled in or out in direction transverse to the bridge. The Scaffold cannot be moved past a panel point unless the Retractable Wings are fully retracted. When the Scaffold is in position between panel points the Retractable Wings, one on each side, may be run out, thus extending the working deck to the inner sides of the stiffening trusses. Movement of the Retractable Wings is effected by means of a rope drive motored by electric power.

Necessary limit switches and interlocking is provided to safeguard the units against damage by improper operation and to limit travel to that intended.

Machinery for driving the Carrier is located on the Shoreward side only and it is connected only to wheels at the Shoreward end of the trucks. Wheels at the opposite end of the trucks are idlers. Rail Clamps are provided adjacent to the drivers only — two per Scaffold.

Interior Scaffolds for Center Span and Side Spans are alike except that the wheels and driving machinery of the Side Span Scaffolds are set lower to compensate for the difference in distance between top of rail and underside of floor beams in the two spans.

Torque wrenches were used to check adjustment of pneumatic impact wrenches used for high tensile bolts.

EXTERIOR SIDE SCAFFOLDS. The Exterior Side Scaffolds are in pairs, one right-hand and one left-hand. They hang in vertical planes just outside the Stiffening Trusses. At the top they are provided with wheels which roll on the Crane Rails previously described. At the bottom the two Scaffolds forming a pair are joined by two wire ropes which restrain them from outward movement.

These Scaffolds are thirty feet long in the longitudinal direction of the bridge and three feet wide. At each end is a latticed upright hanger. At their tops the hangers terminate in trucks which extend inward in cantilever fashion carrying at the cantilevered end a wheel which rolls on the Crane Rail. The truck on the Shoreward



end is designated a "Driver Truck" because its wheel is driven by electric motor-driven machinery mounted on the truck. The truck at the other end of the Scaffold is designated a "Trailer Truck" because its wheel is not used for propulsion. The transverse wire ropes mentioned above are attached to the bottom ends of the hangers — one to each hanger. About eight feet up from the bottom end of each hanger there is mounted thereon a rubber tired wheel which in operation rolls along the outside webs of the bottom chords of the Stiffening Trusses. They serve to restrain the Scaffolds against inward movement. At the upper end of each hanger there is mounted thereon a trolley which functions in a horizontal plane in rolling engagement with the aforementioned trolley beams. These are arranged to hold the wheels in proper lateral relation with the Crane Rails. Rail Clamps located six feet three inches in from the trucks provide means for mooring the Scaffold in any position.

Combined with each Rail Clamp support is a Safety Stop which just clears the rail by a fraction of an inch. This Safety Stop is capable of supporting the Scaffold in the event of any wheel, truck or rail failure.

The outboard or outer plane of the Scaffold is provided with a system of diagonal bracing and the inner face is open except for handrailings and one vertical tension member on the centerline.

The top deck of this Scaffold which is about one foot nine inches above the top chord of the Stiffening Truss serves as an auxiliary operator's platform and a means of access to lower levels. Below the top deck there are four platforms three feet wide and thirty feet long. They are spaced vertically about eight feet apart (the bottom space is greater) to serve as working platforms for maintenance workers.

The ends and back side of the Scaffold are covered with woven wire fabric as a safety measure to prevent men or material from falling off the Scaffold.

Electric power for operating the Exterior Side Scaffolds comes from an engine generator set on the Exterior Underneath Scaffold. This power is delivered by a flexible power and control cable extending from terminal box on each end of the Underneath Scaffold to Cable Reel mounted on the lower end of each Shoreward hanger.

EXTERIOR UNDERNEATH SCAFFOLD. The Exterior Underneath Scaffold comprises a working platform eight feet wide extending the full width of the bridge and supported by two trusses four feet deep. The Scaffold is provided with two trolleys per truss spaced forty-five feet center to center and in rolling engagement with trolley beams attached to the underside of the Track Girders which places them just below the Bottom Lateral Bracing of the bridge.

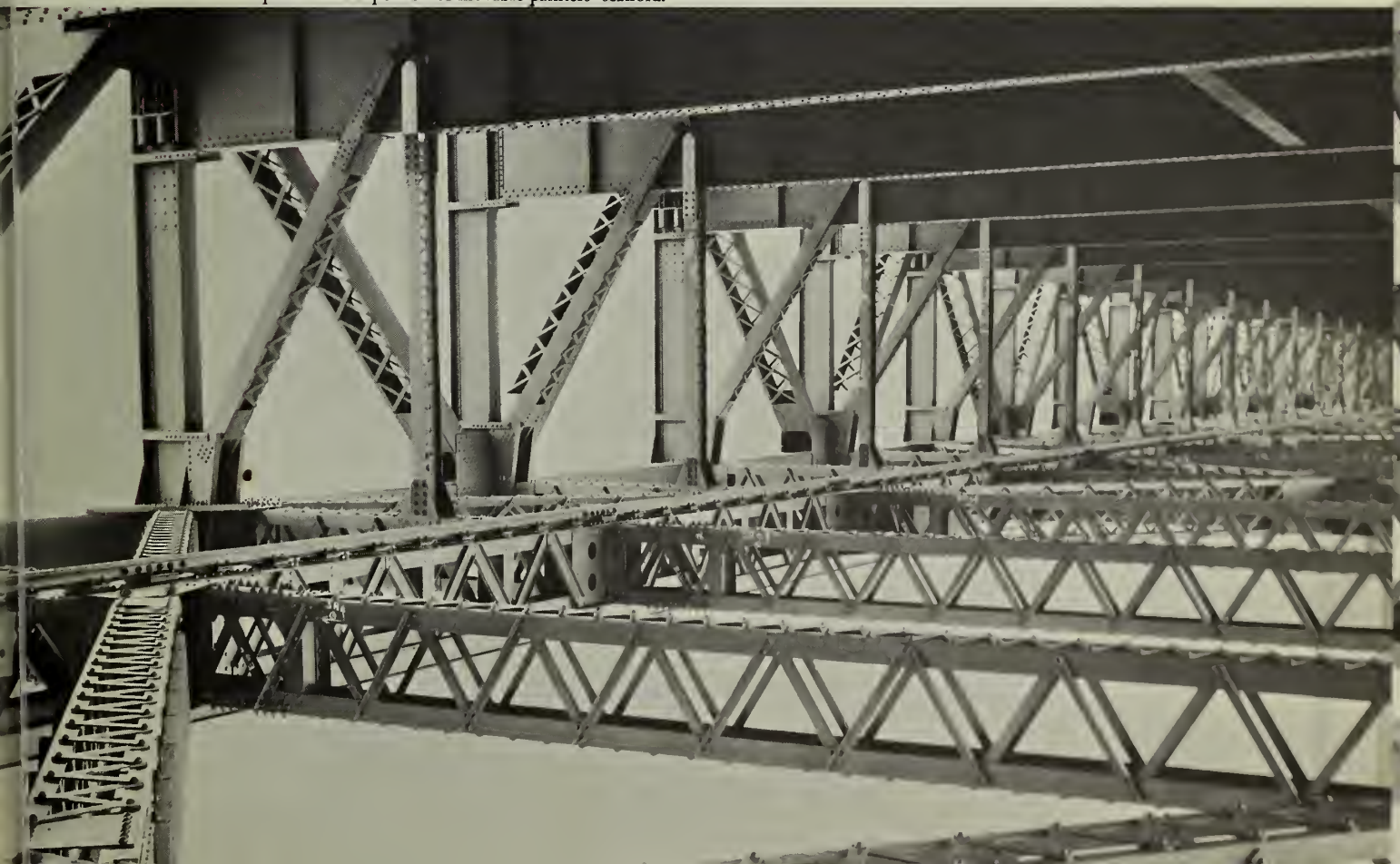
This Scaffold is operated in conjunction with the pair of Exterior Side Scaffolds with which it is associated. It is propelled along the bridge by means of wire ropes working over drums (one at each end). The ends of the wire ropes are anchored to each end of the adjacent Exterior Side Scaffold. The drums are electric motor driven, power being derived from an engine generator set located on the Scaffold.

A latching device is provided at each end which affords a means of mooring the Scaffold to the Exterior Side Scaffolds.



Gusset plate detail of one transverse and two diagonal struts connecting to bottom chord of stiffening truss occurs each 100 feet at panel points chosen to avoid main gusset connections for truss diagonals.

Track girders, 45 feet center to center, were final elements to be erected. Permanent tracks supported inside portion of movable painters' scaffold.







Connection plate where both transverse and diagonal struts meet bottom chord of stiffening truss.

Connection of track girder to transverse strut occurs every 25 feet, and to diagonal every 50 feet.





Exterior side scaffold serves here as working platform for personnel. Exterior underneath scaffold, located in this view beneath a transverse strut, extends full width of suspended deck structure.

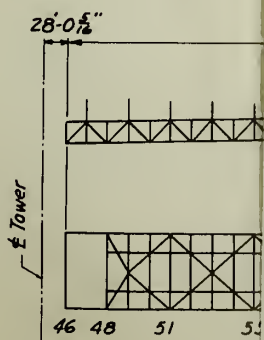
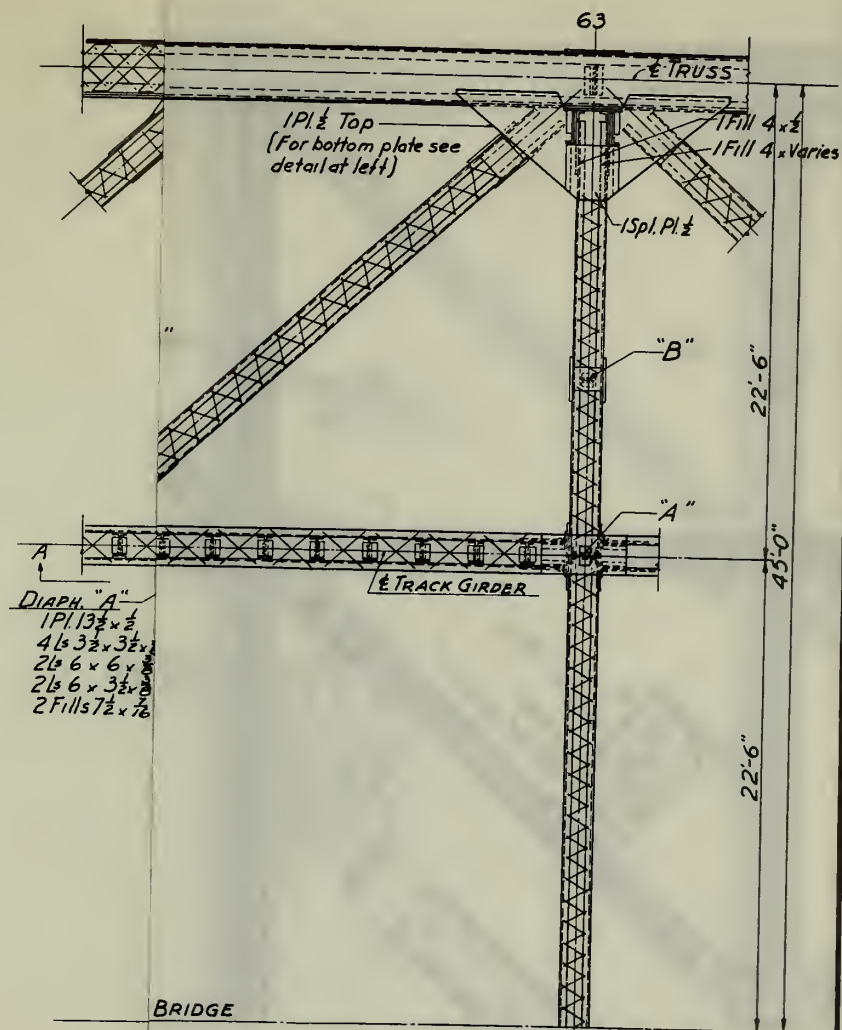
Interior movable scaffold is supported on track girders. Retractable wing section is shown fully extended to provide close working platform.

Exterior side scaffold number 4 is shown approaching Marin cliffs with tarpaulin wind break removed.





PLATES



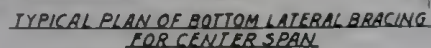
THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

REPORT OF BOARD OF ENGINEERS
JANUARY, 1953

OTHMAR H. AMMANN CHARLES E. ANDREW
CLIFFORD E. PAINE

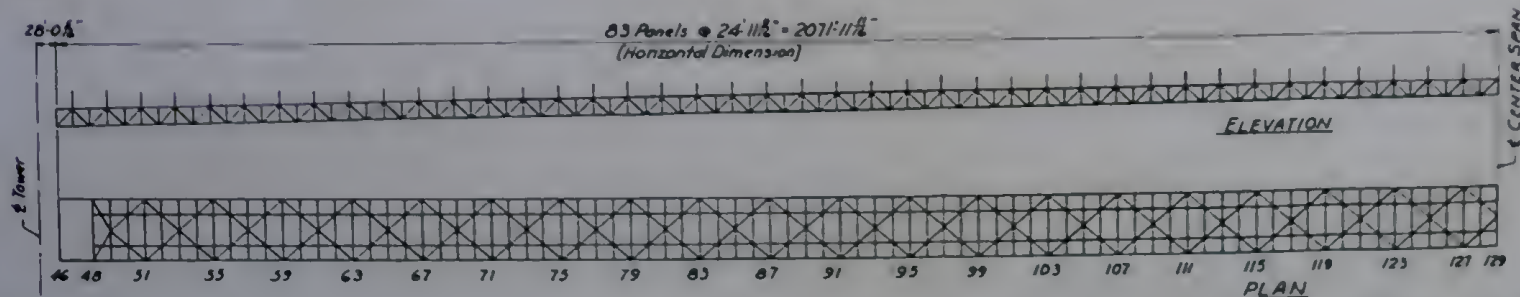
PROPOSED BOTTOM LATERAL SYSTEM
— PLAN —

SCALE IN FEET
0 1 2 3 4 5 6 7 8 9 10



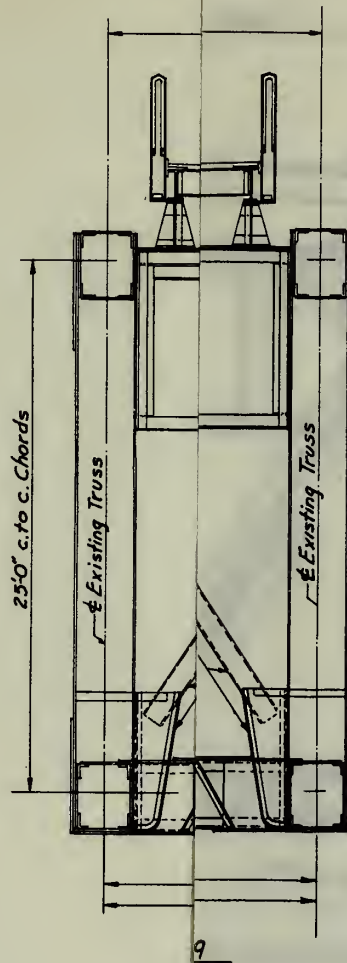
ALL TRACK DIMERS
4Ls 7 = 4.8
Drops 2 = 16.75"
Verticals 1 = 14W 53"
DL 28 = 2
Tie Pts 22 = 8 Top & Bottom

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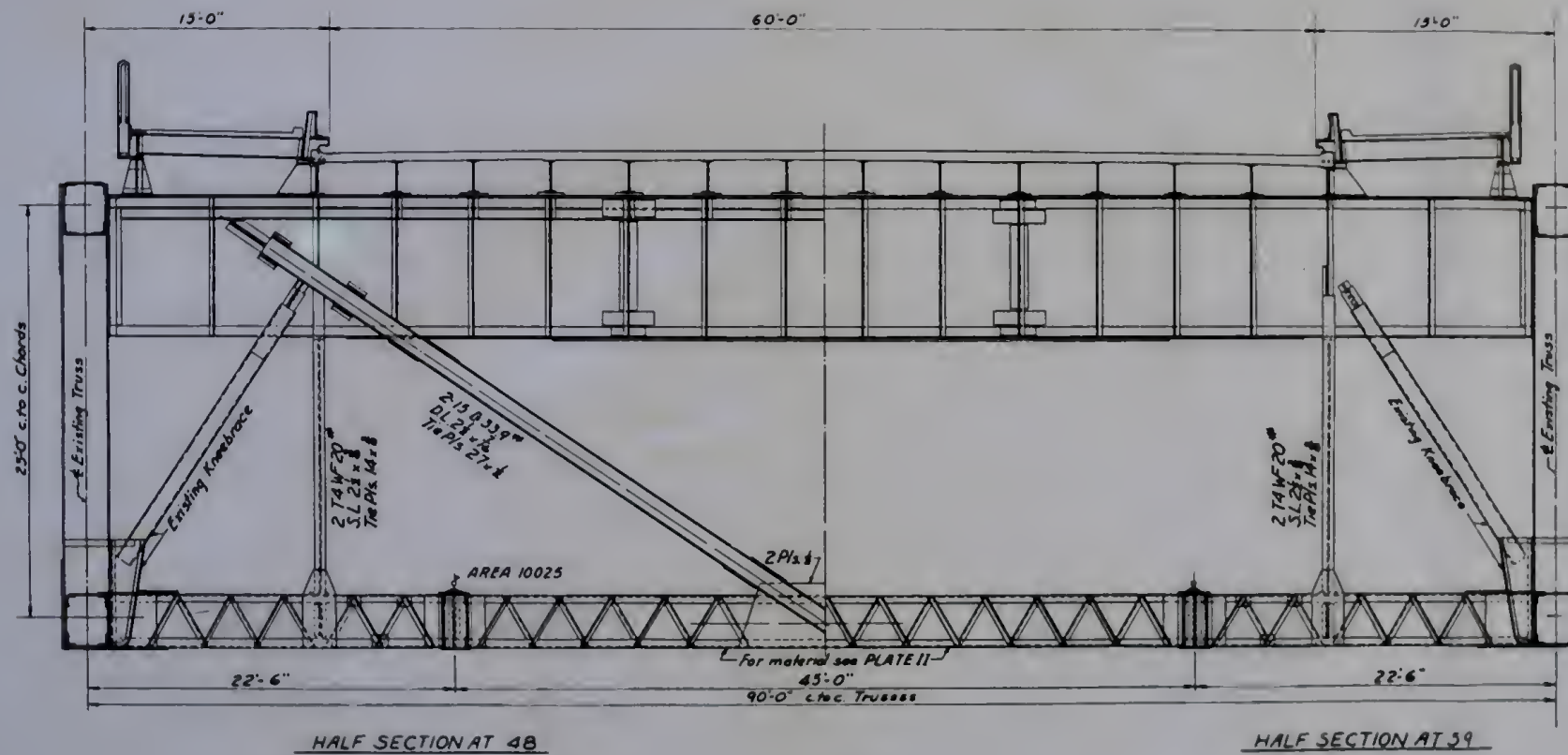
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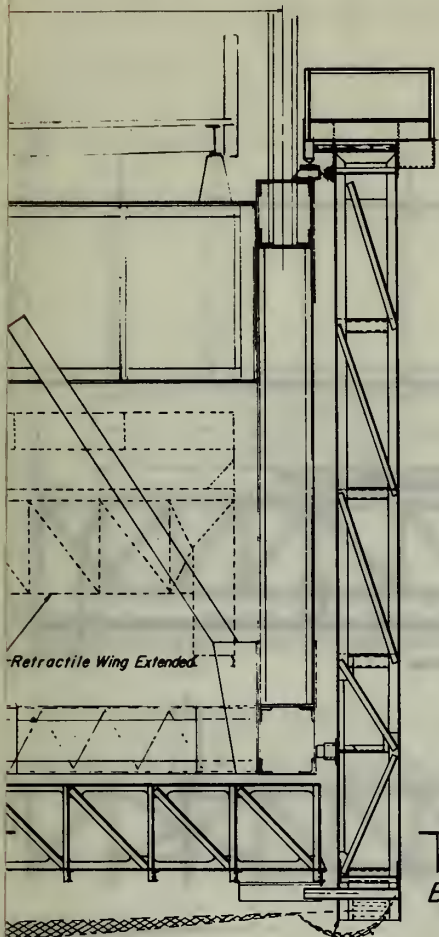
PROPOSED BOTTOM LATERAL SYSTEM
-SECTIONS-

SCALE IN FEET
0 1 2 3 4 5 6 7 8 9 10

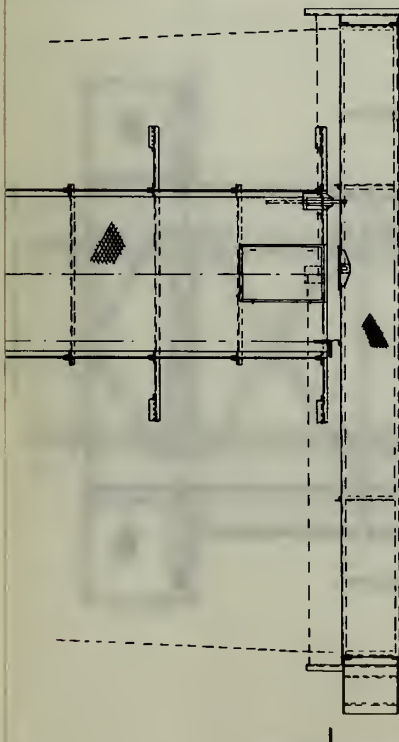


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 PROPOSED BOTTOM LATERAL SYSTEM
 -SECTIONS-
 0 1 2 3 4 5 6 7 8 9



EXTERIOR SIDE SCAFFOLD-ES2^R



TRAVELING SCAFFOLDS

FOR THE
GOLDEN GATE BRIDGE & HIGHWAY DISTRICT
OF CALIFORNIA

—GENERAL DRAWING—

CLIFFORD E. PAINE — CONSULTING ENGINEER
FENNIVILLE, MICHIGAN

APPROVED
GOLDEN GATE BRIDGE & HIGHWAY DISTRICT
GENERAL MANAGER

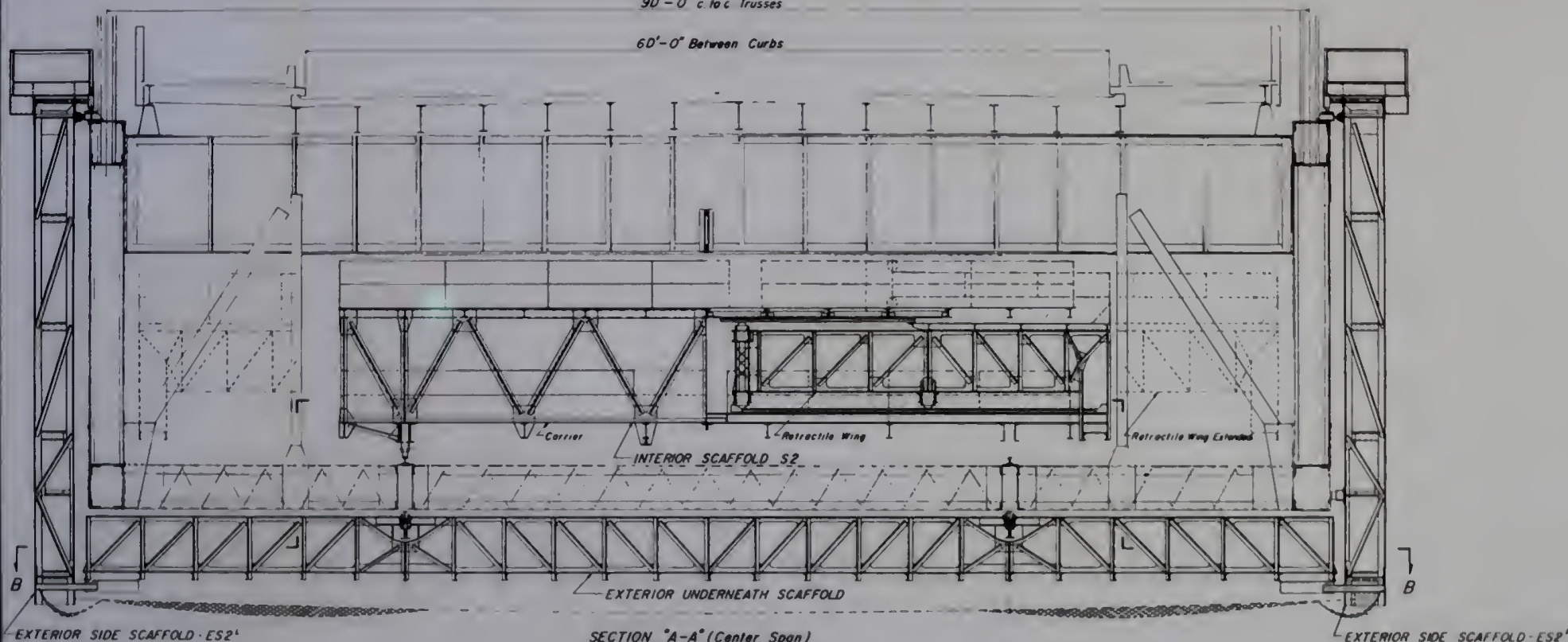
DATE — Mar. 25, 1955
SCALE — 1/4" = 1'-0"

SHEET NO. 2

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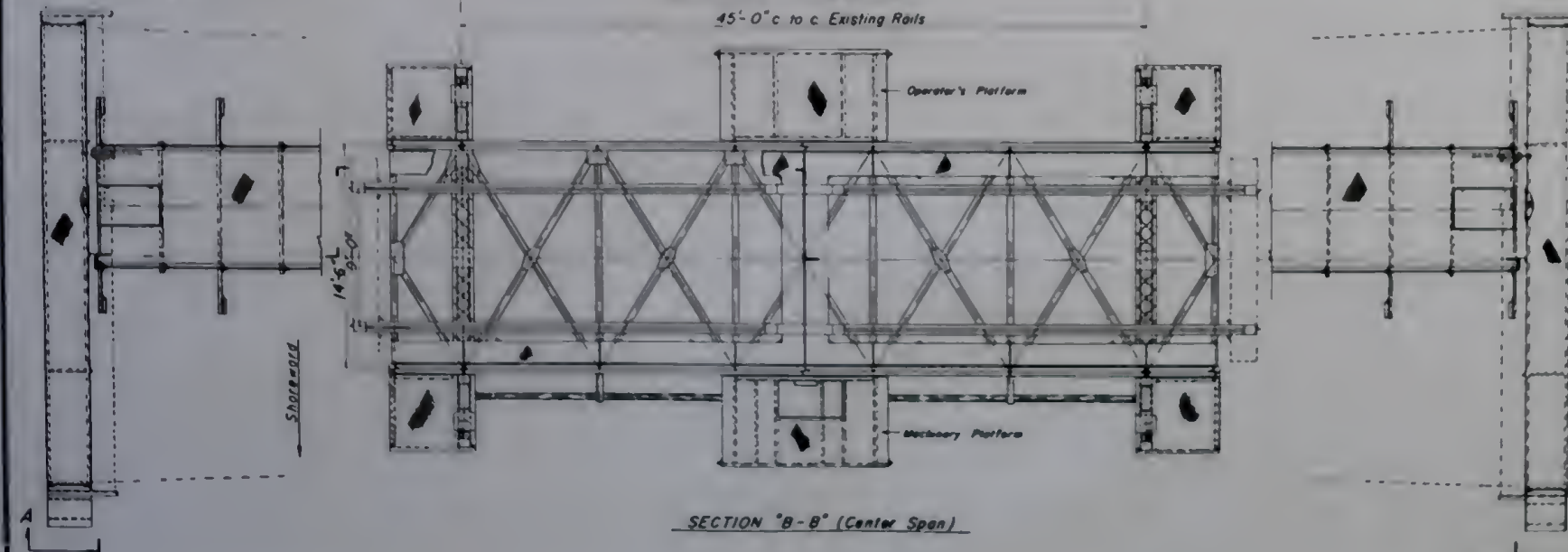
90'-0" c to c Trusses

60'-0" Between Curbs



SECTION 'A-A' (Center Span)

45'-0" c to c Existing Rails



SECTION 'B-B' (Center Span)

TRAVELING SCAFFOLDS
FOR THE
GOLDEN GATE BRIDGE & HIGHWAY DISTRICT
OF CALIFORNIA

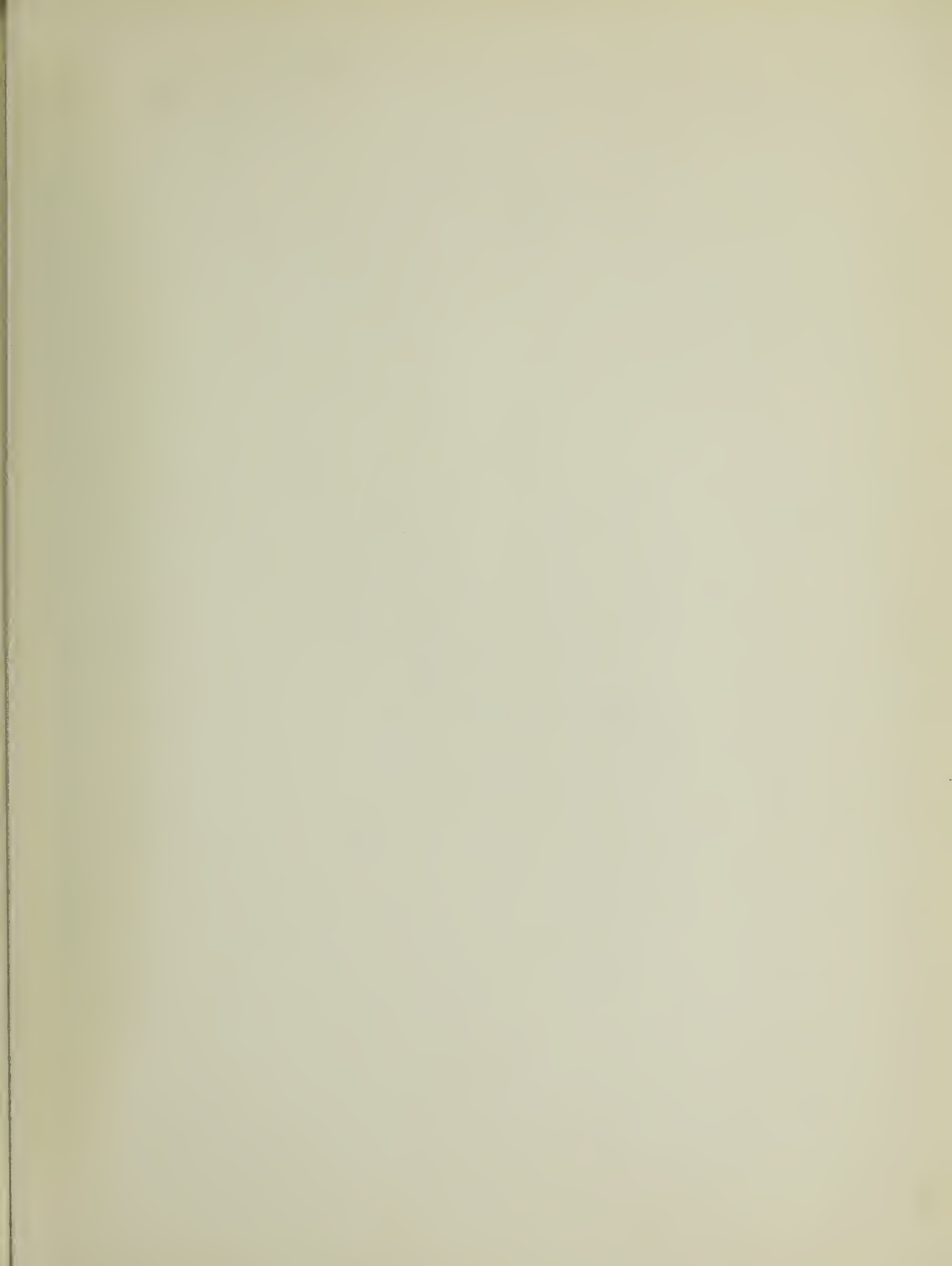
- GENERAL DRAWING -

CLIFFORD E. PARR - CONSULTING ENGINEER
FREMONT, CALIF.

APPROVED
GOLDEN GATE BRIDGE & HIGHWAY DISTRICT
GENERAL MANAGER

DATE - Mar. 23, 1923
SCALE - 3/4" = 1'-0"

SHEET NO. 2



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